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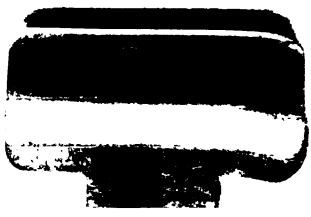

STATE OF NEW YORK.

DEPARTMENT

OF THE

STATE ENGINEER AND SURVEYOR.

GENESEE RIVER STORAGE SURVEYS.

1.— Report of Martin Schenck, State Engineer and Surveyor.
2.— Report of E. Sweet, as Consulting Engineer.
3.— Preliminary Report of Geo. W. Rafter, Engineer in Charge, to Martin Schenck, State Engineer and Surveyor.
4.— Final Report of Geo. W. Rafter, Engineer in Charge, to Campbell W. Adams, State Engineer and Surveyor.

ALBANY.
JAMES B. LYON, STATE PRINTER.
1894.

REPORT

ON THE

GENESEE RIVER STORAGE

BY THE

STATE ENGINEER AND SURVEYOR

OF THE

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REPORT.

Genesee River Storage.

One of the provisions of chapter 726, Laws of 1893, makes an appropriation of $10,000 to be expended by the State Engineer and Surveyor for determining the advisability and desirability of carrying out the plan recommended by the commissioners, Evan Thomas, John Bogart and Chas. McLouth, in their report to the Legislature of February 2, 1893, on the subject of improving the supply of water from the Genesee river to the Erie canal, and for making such maps, plans, estimates and recommendations in connection therewith as he may deem desirable. In carrying out the purposes of this law, I organized an engineering party, under the charge of Mr. Geo. W. Rafter, C. E., and entered upon a critical and extensive examination of the conditions existing here. I have also held frequent consultations on the engineering questions involved with the Hon. Elnathan Sweet, C. E., and submit herewith the reports handed me by these two gentlemen. As the questions concerning the flow of the stream, the pondage to be obtained by dams, of various heights at different sites, and the amount of land to be overflowed by the proposed dams, have been so ably and exhaustively treated by my predecessor, Hon. John Bogart, in his report to the Legislature for the year ending September 30, 1890, and the appendices thereto, I have considered it useless to investigate them further, but accepting his conclusions on these questions, have confined my examinations to local conditions affecting the construction of dams at the three sites herein referred to, especially as to the character of the foundations, the means of transportation for the large quantity of material entering into a work of such magnitude, the probable source from which the needed stone, sand, cement, clay,

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etc., can be drawn, the proper dimensions and locations of dams in the proximity of sites Nos. 1 and 2, referred to in the report of the State Engineer and Surveyor for the year 1890, and the report of the commission of 1893, and also of a dam on a new location, designated herein as the "hog back" location (but which has not been previously considered), with their spillways, regulating works, etc. I have also made preliminary study of the different forms of dams which I consider may safely be recommended for construction at the different sites proposed; for first, a dam fifty-eight feet high; second, a dam fifty-eight feet high, but so designed as to be capable of being increased to the height of 130 feet; third, a dam 130 feet high, built continuously; and fourth, raising the fifty-eight feet dam (No. 2) to a 130-feet dam (No. 3), and herewith submit tracings showing these different plans.

On site No. 1 it is proposed to erect a stone dam having a spillway over its top. As it is desirable to reduce to a minimum, consistent with proper economical considerations, the depth of the sheet of water passing over it in times of excessive flow, I do not think the spillway should be of less length than 400 feet and the face work of this portion of the structure should be built of granite. The remainder of the exposed portion of dam would probably best be built of Pennsylvania sandstone, while its backing could be composed of stone found at Portage, Nunda or possibly even nearer the site of proposed work. In order, however, to protect the base of the structure from the eroding action of the water, which in time of flood will pass over its crest, it will be necessary to obtain a water cushion, immediately below the structure, not less than thirty feet deep. This can be accomplished by the building of a subsidiary dam somewhere between proposed site of structure and Mt. Morris, to a height of fifteen or sixteen feet above present surface of river. No apron or other protection would be required for this subsidiary dam, as it would be founded on rock at an elevation of fifteen feet below river which would not be eroded by a water fall of sixteen feet. Another advantage gained by this subsidiary dam is the increased

The Genesee Valley Just Below the Village of St. Helena.

pondage it affords. In this structure, gate-house and regulating works would necessarily become a portion of the dam itself.

At site No. 2 it is also proposed to erect a masonry dam, but here the topographic conditions admit of the construction for the 130-feet dam of a spillway 400 feet wide, entirely independent of the dam itself, cut out of the shale rock which forms the cliff and protected by stone side-walls and concrete bottom. As, however, the foot of this spillway would be located something less than 100 feet from the toe of the dam, it would be necessary to provide here as at site No. 1 a thirty-feet water-cushion. In this construction only a small amount of granite would be needed for gate-house and trimmings. Otherwise materials required will be similar to those needed at No. 1. For a fifty-eight-feet dam here, if the water cushion, referred to, is provided, the spill could probably be safely taken over the crest of the dam. In this construction, as at site No. 1, the gate-house and regulating works would necessarily be built as a portion of the dam itself.

At the "hog-back" location it is proposed to build an earthen dam with a masonry core. The earthen dam is to be raised to a height of twenty-five feet above ordinary level of water and a spillway obtained by cutting through the narrowest portion of the remarkable ledge of rock which exists here and which it almost looks as though nature had provided for this purpose.

Neither my predecessor nor the commission of 1893 seriously considered the location now proposed, for the reason that a superficial examination of this hog back raised some doubts as to whether or not it was sufficiently strong to withstand the pressure created by a high dam at this point. The very careful and extended surveys that I have had made here and the experiments described in Mr. Rafter's report of pumping water, under pressure, into holes drilled for the purpose, remove any doubt on this subject which could have heretofore existed. The advantages of this location are, first, that it removes the spillway entirely from the dam, and thus renders it impossible that, with a spillway of proper width and crest of dam placed at proper elevation, any damage

can be done to the dam by the most extreme freshet; second, it renders it possible and convenient to erect gate-house and regulating works entirely independent of the dam; and, third, the proposed structure can be built at from thirty to forty-five per cent less cost than at site No. 1.

While I know a prejudice exists against the building of high dams, either partially or entirely, of earth, I am of the opinion that where ample spillway facilities can be obtained completely independent of the earthen structure, and where, as in this case, rock walls exist, against which it is entirely feasible to make water-tight connections at the ends of the dam, this prejudice is entirely unfounded. By building a dam on the site here proposed, and erecting a spillway where I have designed it, the channel of the stream will be so changed that the dam will form a lake, and thus become protected from wind action and from the force of the current.

On all the sites heretofore treated of, a shale rock underlies the river-bed at various elevations, averaging about fifteen feet below the normal surface of the river. Our borings and experiments made by pumping water into holes indicate that upper portion of the rock is somewhat porous in its nature. The plan which I have prepared, therefore, contemplates the digging of a trench to a depth of from fifteen to thirty-feet below the rock surface (or thirty to forty-five feet below the ordinary elevation of water in the river) and the filling of this trench with impervious concrete masonry. With this treatment, aided possibly by putting in small quantities of grout below the elevations heretofore referred to, I doubt not that all danger of leakage under base of dam will be obviated. Based upon the plans and principles heretofore outlined, I estimate cost of these various structures as follows:

For fifty-eight feet dam, at site No. 1.....	$1,200,000
For fifty-eight-feet dam, at site No. 2.....	1,300,000
For fifty-eight-feet dam, at hog-back site.....	850,000
For a fifty-eight-feet dam, built of dimensions below its crest rendering it practicable to raise the same to a height of 130 feet:	

A General View of the Hogback and Horseshoe Bend.

At site No. 1	$2,250,000
At site No. 2	2,000,000
At hog-back site	1,400,000
To build a dam 130 feet high:	
At site No. 1	3,150,000
At site No. 2	3,350,000
At hog-back site	2,100,000
To first build a dam fifty-eight feet high, and afterwards raise it to 130 feet:	
At site No. 1	$3,250,000
At site No. 2	3,400,000
At hog-back site	2,500,000

A study of the plans and estimates convinces me that, while it would be very desirable to obtain, as at site No. 2, a spillway entirely independent of the structure itself, that the advantages to be gained thereby would not be commensurate with the increased cost and loss of pondage of this structure over that at site No. 1, and, therefore, that the choice of location has narrowed down to between site No. 1 and the hog-back site.

The above estimates are based upon the presumption that stone for concrete will have to be obtained from quarries at a considerable distance from the proposed work, and those contemplating a stone structure presuppose the use of quarried stone in all its parts, except some of the minor foundation work. It has been suggested that, by the use in concrete of the shales forming the cliff against which these dams abut, the cost of these structures may be materially reduced. In order to determine the value of these suggestions, I have had made a number of blocks of concrete, composed of stone, whose durable qualities and strength are well known, and of the shales in question, mixed with varying proportions of sand and cement, and these blocks will hereafter be subjected to such tests as are deemed necessary to determine the questions involved. Until these tests are made, no intelligent conclusion can be arrived at as to the desirability and economy of the proposed use of this shale.

The figures given by Mr. Bogart and adopted by the commission of 1893, seem to show pretty conclusively that a dam fifty-eight feet high is needed here for the best interests of the Erie canal in its present condition. The construction of a dam of this height therefore would develop little or no benefit to interests other than those of the Erie canal, as in order to meet the canal requirements upon this pond it would be necessary to pretty constantly maintain the elevation of water up to the height of the crest of the spillway. If, therefore, this work is entered into not only for the purpose of supplying the present needs of the canal, but also its prospective needs, and for generating and forming a water storage which could be applied to other interests, I would strongly advise the building of a dam 130 feet high, which approaches the absolute control of the stream and secures to a large degree the full benefit of this most valuable resource. I have considered the problem of building such a fifty-eight-feet dam now, as it would be possible to heighten in the future to the 130-feet limit, but the difficulties of construction on this basis, as referred to in detail by Mr. Rafter, as well as the largely increased cost of the final securing by this means of the 130-feet dam, render it, in my opinion, very desirable that if this work is undertaken, it shall be prosecuted at once to completion.

It is possible that the 130-feet dam could be built in from two and one-half to five years, dependent upon the location and style of structure decided upon.

The advantages to be derived from the increased supply of water from the Genesee river, not only to the canal system of the State, but to a large number of its citizens living in the counties of Monroe and Livingston, are so self-evident as to scarcely require argument. The questions to be determined are, therefore: First. Are these advantages commensurable with the cost of obtaining them? Second. Is it not proper in the present favorable financial status of the State treasury, and cramped financial condition of many of our laboring and other citizens, for you to authorize at this time an expenditure of the funds necessary to realize these advantages? and, Third. If these two questions be answered in

the affirmative, what proportion of the cost should be borne by the State and what by other parties interested in this improvement?

At an early period in the history of the canals the Genesee river was considered an important source of water supply for the canal, both easterly and westerly of Rochester, and the question of appropriating this supply was the subject of a document signed by Governor DeWitt Clinton, then acting canal commissioner. If at that early date the necessity for an increased supply was apparent, there is little doubt that such a necessity exists to-day.

That portion of the Erie canal now fed almost entirely by the waters of Lake Erie, extends sixty miles eastward from the terminus of the Genesee river feeder, and it can scarcely be doubted that such decreased velocity of water in the canal prism between Buffalo and Rochester, a distance of ninety miles, as would be obtained by an abundant well-regulated supply from the Genesee river would be of vast benefit to canal navigators.

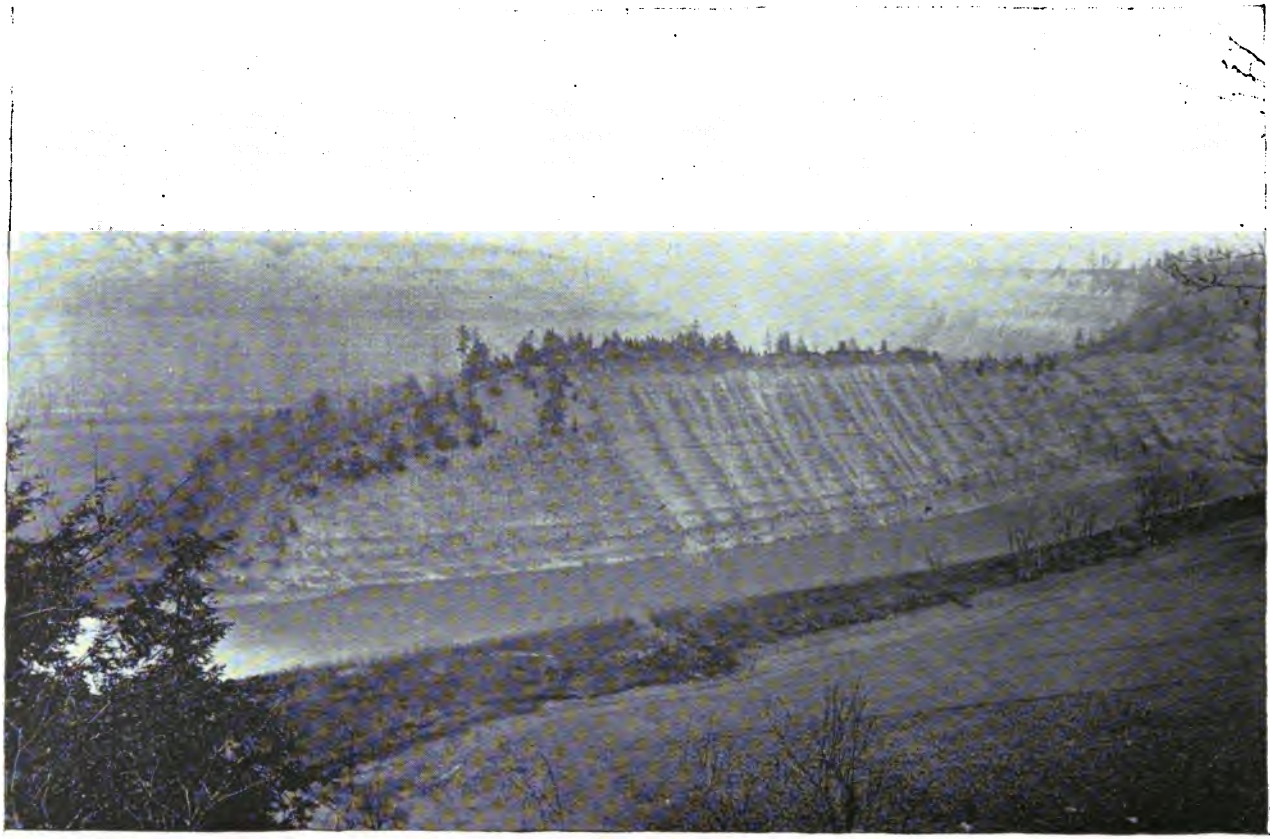
While it may be possible that no actual necessity has, as yet, arisen for this increased supply, the advantages of the decreased velocity thus obtained and the increased facilities for feeding the canal in the spring, and the refilling of it after a break, the maintaining, during a break, of navigation on a long portion of the canal, which has now to be entirely abandoned until the close of the break, are so great as to render it probable that true economy would dictate the immediate prosecution of the work. Added to this are certain equitable considerations which should be given their proper weight. It can scarcely be disputed that the State has appropriated such of the waters of the Genesee river as are necessary to the proper interests of the canal. In this condition of affairs, the first title to the water vesting itself in the State, it can hardly be expected that any corporation can be formed for the development of this power unless the interests of the State can be extinguished. I do not think that proper public policy would allow the yielding by the State of this or

any other of the important water privileges it now owns; yet, upon the other hand, I doubt the wisdom of a policy by the State, which, while refusing to surrender its rights would also refuse not only to develop them itself, but to allow any one else to develop this great source of wealth.

I have been advised by responsible citizens of Rochester, that if the rights of the State in the premises could be wiped out, there would be little difficulty in organizing a private corporation to press this work forward to a successful completion. Another element in the consideration of this question is the fact that the State has heretofore acknowledged, by its action in building reservoirs on the Black river, the equitable obligation resting upon it to make restoration "in kind," of waters diverted from private owners for public use.

Still another consideration entitled to weight in deciding a question of such far-reaching and vast importance as this, is the agitation now before the public looking toward the securing of means for electrical propulsion of canal boats. While an investigation and study of this question has not rendered it by any means certain that this method of propulsion will become shortly one of fact and not of expectation merely, I think it may be fairly stated that there is a reasonable hope of the economic adoption of this system on our canals, and if these hopes can be realized, the power generated here by water passing from this reservoir through a number of turbine wheels, suited to the varyings heads under which they would act, would supply a force which would largely aid us in solving the question of cheap canal boat propulsion, and this would be true whether the success of the electrical system should be obtained by means of trolley wires or storage battery. If it came in the latter direction double advantage would be derived, the daily flow of the stream being devoted to manufacturing purposes, and night flow to the storage of the electrical fluid for canal purposes.

In concluding my report upon this subject I desire to give expression to my realization of the valuable assistance I have received in these investigations by the very comprehensive and



A General View of the Hogback from the West High Bank.

able report made to you by Mr. Bogart in 1891; the valuable analysis of this report and the recommendations made by the commission of 1893; the intelligent and well-directed efforts of Mr. Geo. W. Rafter, the engineer assigned to the charge of the work done during the past summer, to the sound advice given and suggestions made by Hon. Elnathan Sweet, whom I have freely consulted in this matter, and to the assistance given me by Mr. Chapman L. Johnson, Deputy State Engineer and Surveyor, to whom I have confided the charge of the detailed work connected therewith.

REPORT OF E. SWEET.

ALBANY, N. Y., *December* 23, 1893.

Hon. MARTIN SCHENCK, *State Engineer and Surveyor:*

Sir.—Having at your request twice visited the gorge of the Genesee river above Mt. Morris and inspected the various sites suitable for the erection of a dam to impound the freshet water of that river, and given some attention to the collection of engineering data, relative to a proper selection of site and a determination of the probable cost of accomplishing a comprehensive plan for storage of these waters, I beg leave to offer such suggestions as the hastily collected results of the very thorough surveys, borings and explorations which you have just finished there present to my mind.

The preliminary surveys and studies of this subject made by your predecessor in office give a very clear and comprehensive idea of the general scope and limitations of the project of regulating the flow of this stream.

These studies and surveys as outlined in the appendices to his report to the Legislature in 1890 show the storage capacity to be created in this gorge by the erection of dams of varying heights, and the probable increase in outflow they would severally be expected to produce from rainfall and run-off tables prepared from statistics gathered in the Genesee watershed.

This report also treats exhaustively the question of the present wants of the canal, and shows clearly the need of 1,500,000,000 cubic feet of water to be stored on this stream to properly and safely operate the canal in its present state, which, of course, fixes the lowest limit of storage which the State's interest requires and the future wants of the canal, as well as the vast interests of the large population affected by the regulation of this stream, make a much larger volume of storage desirable.

West End of Hogback

These preliminary surveys give the capacity of the storage basin to a height of 130 feet above the stream bed at the two proposed sites for dams near Mount Morris, and I think you have wisely made your detailed surveys and exploration of sites broad enough to cover accurate data for estimating the difficulties and the cost of constructing dams to store water to that or any lower height and confined them to the two first sites described by Mr. Bogart and the one immediately above his second location at the sharp bend, known as the "Hogback."

The fifty-eight foot reservoir affords very slight relief to the stream flow, as it must be held full for canal contingencies, while the 130-feet reservoir increases many times the natural flow of the stream and gives nearly seventy-five per cent of the beneficial effects of a completely developed regulation of its flow, which latter is not reasonably practical.

Estimates have been prepared for erecting dams at each of these sites on the following bases: First, the cost of a dam fifty-eight feet high, with proper regulating apparatus for permanently holding water to that elevation only; second, the cost of a fifty-eight feet dam, so founded and built as to be capable of being raised to store water to a depth of 130 feet; third, the cost of subsequently raising the dam from the fifty-eight feet level to that of 130 feet, and, fourth, the cost of building at once a dam 130 feet high.

A careful consideration of the quantities and value of the various items of work involved in the utilization of these various sites leads me to believe that a reservoir with suitable fifty-eight feet dam, can be built on the first basis of estimation above described, at location No. 1, for $1,200,000; at location No. 2, for $1,300,000; at the "Hogback," for $850,000. On the second basis of estimation, fifty-eight feet reservoir, at location No. 1, $2,250,000; at location No. 2, $2,000,000; at "Hogback," $1,400,000. To raise the latter to 130 feet will cost, at location No. 1, $1,000,000; at location No. 2, $1,400,000; at "Hogback," $1,100,000. To build at once a dam 130 feet high, will cost, at location No. 1, $3,150,000; at location No. 2, $3,350,000; at "Hogback," $2,100,000.

Of the three sites, location No. 1 gives nearly ten per cent the greater storage capacity, at about ten per cent less cost than location No. 2.

The advantage of site No. 2 is, that it affords an opportunity, and the estimate provides, for a spillway cut through the rock, independent from the dam itself, but I do not consider this a sufficient ground for disregarding the greater storage and greater economy afforded by site No. 1.

I am favorably impressed by the conditions developed by the surveys and borings at the "Hogback" location. This anomalous topographic feature, consisting of a rock ledge, extending nearly 2,000 feet at right angles to the axis of the stream, lends itself admirably to the location of a spillway and regulating works, while the gorge opposite its extremity is very little wider than at site No. 2.

The estimates presented for this location assume a dam built of earth, with a masonry core.

This is a very high embankment, but it must be remembered that its location is peculiarly fortunate for such a construction, as the long sharp bend in which it will stand protects it from the waves, while the artificial spillway diverts the channel and the current a thousand feet from its extremity.

It is a far less bold construction than the earthen part of the new Croton dam now building by the city of New York.

For the fifty-eight feet reservoir its superiority is undoubted, and I believe it worthy of very mature consideration for the higher elevation.

Among the considerations affecting the proper attitude of the State towards this project, the equitable rights of the riparian owners on the stream seem to my mind of great importance.

The Genesee is a stream characterized by steep slopes from which the rainfall rapidly runs off, and though it has a large watershed the water runs off in freshets, leaving a small and unreliable summer flow.

It thus follows that, though the river falls more than 200 feet in the large manufacturing city of Rochester, its intermittent flow renders the water power comparatively useless.

General View of Plant as Used at the Hogback.

The only remedy for this is the regulation of the flow by storage, and, as the State long ago appropriated the waters of the Genesee, private enterprise is unable to protect these great interests because its equitable rights are in the grasp of the sovereign.

The Legislature has, however, recognized riparian equities in the case of the Black river, by creating reservoirs on the Moose and Beaver rivers, from which the canal could by no possibility receive water, for the express purpose of indemnifying the mill owners on the Black river for water diverted from that source to the canals.

There seems a still stronger reason why it should favor storage in the Genesee valley, a large part of which is now needed and much more likely to be required in the future for its own purposes.

Very respectfully,

(Signed.) E. SWEET.

Report of Geo. W. Rafter, Engineer in Charge.

ALBANY, N. Y., *December* 15, 1893.

Hon. MARTIN SCHENCK, *State Engineer and Surveyor:*

Sir.—I herewith present the following preliminary report in relation to the Genesee river storage project, on which, as per instructions, I began work July 25, 1893:

The excellent general surveys and studies made under the direction of the Hon. John Bogart, by Messrs. Bailey and Kibbe, and which appear together with the report of Mr. Bogart himself on this project in the annual report of the State Engineer and Surveyor for the fiscal year, ending September 30, 1890, have furnished the basis of the present year's work and no attempt has been made to go over any portion of the ground covered by these engineers. This season's work has been of an entirely different nature from that accomplished by these gentlemen in 1890. The preliminary studies having demonstrated not only the practicability but the desirability of a large storage dam on the Genesee river in the vicinity of Mount Morris, the problem then presented itself in the following form, namely, having given a number of sites where, as indicated by the preliminary surveys, large storage dams can be constructed, it is reguired to determine at which point the largest amount of storage can be obtained at the least expense, the form of construction to be such as will give, so far as engineering skill can forsee, an absolutely safe dam.

The amount of storage obtainable for various heights of dam is exhibited in the report of Mr. Bailey, as already referred to in the annual report for 1890, and the matter is accordingly further

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narrowed to a detailed examination of the available sites, with a view of determining (1) the most desirable site; (2) the form of dam which will best fit the existing conditions; and (3) the cost of a dam, combining a maximum of efficiency and safety with, so far as possible, a minimum of expense.

The main questions needing further consideration in connection with the Genesee river storage project having been thus saliently defined in the foregoing, we may, in order to get our information all together, properly review briefly the main points of the preceding reports.

(1.) In appendix F to the annual report for 1890, Mr. Bogart presents, in response to a resolution of the Senate of March 21, 1889, a statement of (a) why the interests of the State would be subserved by the construction of a storage reservoir on the Genesee river; and (b) an estimate of the amount of storage required for immediate canal purposes.

(2.) Following Mr. Bogart's report appears the report of Mr. Bailey, giving the contents in cubic feet and the area in acres of a pond formed by dams, at each of four feasible locations, for every ten feet increase in depth up to 130 feet.

(3.) Also following Mr. Bogart's report appears the report of Mr. Kibbe, which includes (a) a short history of the Genesee river feeder at Rochester, and the relation of the present available water supply from the Genesee river to the proper maintenance of navigation in the Erie canal; and (b) a brief consideration of the hydrologic questions involved.

(4.) Under a concurrent resolution of the Legislature of 1892 a commission, consisting of Evan Thomas, Esq., president of the New York Produce Exchange, Hon. John Bogart, and Judge Charles McLouth, of Palmyra, was appointed "to inquire, consider and report what measures may be expedient for the State to adopt for carrying into effect the suggestions and conclusions contained in the special report of the State Engineer and Surveyor on the supply of water from the Genesee river to the Erie canal." This commission inspected the Genesee gorge, and gave a number

of hearings, at which were present gentlemen representing the various interests involved, and after due consideration of the main and cognate questions, reported:

(a) That the project of building a large storage dam on the Genesee river is entirely feasible.

(b) That the control of the waters of the Genesee river resides wholly in the sovereign power of the State, and that hence the State has the right to draw therefrom at its discretion and whenever it chooses any such amount as it may deem necessary for public purposes.

(c) That it is not only practical to determine the maximum quantity which the State may require from the Genesee river for the use of the Erie canal, but that this has been to great extent done in the annual report of the State Engineer and Surveyor for 1890.

(d) That it will be largely to the interest of the State to provide storage on the Genesee river to the amount of 1,500,000,000 cubic feet for canal purposes.

(e) That such storage be provided at Dam Site No. 1, a distance of one and twenty-two one-hundredths miles above Mount Morris.

(f) That the water now taken from the Genesee river for canal purposes is of great value to the State and its loss would be a corresponding deprivation.

As the result of the examination the commission recommended:

(a) That a dam fifty-eight feet in height be erected on the Genesee river at Dam Site No. 1.

(b) That the same be built of sufficient strength in base, sidings, section and vertical rise to allow it to be extended to 100 feet in height.

(c) That it be built with proper gates, and apparatus for drawing off the water from the pond created above it, to any desired water line, and that in seasons when navigation is not in progress this be for the use and benefit of citizens owning or operating plants by means of hydraulic power between said dam and Lake Ontario — along the line of the river — and so as to afford for such uses a continuous and regular flow; and that it be at all

times under the charge and control of the proper officers of the State; that, under such charge, during the season of navigation, like opportunities as may not interfere with the supply demanded by the State may also be afforded said citizens.

(d) That the said citizens may extend the dam to the height of 100 feet, with proper gates and apparatus for drawing down the water line of the pond formed by the said dam to said fifty-eight feet, and shall have the execlusive privilege to do so at all times.

(e) That said vertical rise above the fifty-eight-foot line be constructed with the approval and subject to plans of the proper officers of the State.

(f) That the State be released and indemnified from expense or damage on account of such building above the crest of said fifty-eight-foot dam.

The commission concludes its report with a short discussion of the relations of the State to the owners of hydraulic powers on streams which have been declared public highways.

Having thus briefly defined the present status of the Genesee river storage project we may now proceed to the special work in hand.

The general examinations made in 1890 indicated five available dam sites where large storage can be obtained in the Genesee gorge between Mount Morris and the lower falls of Portage. Four of these have been designated in the 1890 report as Dam Sites Nos. 1 2, 3 and 4. The consensus of opinion of all who have studied the Genesee gorge is that Dam Sites Nos. 3 and 4 are far less desirable than the lower sites by reason of a greatly decreased storage for a given height of dam. All the engineers who have thus far examined the project are agreed on this point. They are also agreed, it is believed, that a given expenditure will yield better results in storage obtained by building a high dam at some one of the lower sites than can be obtained by one, two or three small dams higher up.

In this report the same designations will be given to the dam sites as are used in the 1890 Annual, and for the locations of the several sites from 1 to 4 inclusive, reference may be made to Mr.

Bailey's map accompanying his report therein contained. The discussion, however, is entirely confined to Dam Sites Nos. 1 and 2 and the Hogback location, which latter has not been discussed in the previous reports.

As preliminary to a thorough study of the problem in hand a detailed topographical map of the territory in the vicinity of the several sites to be compared was imperatively necessary. The making of such a map of the gorge was a matter of severe labor, involving as it did the taking of 120 cross-sections. Many of these are of banks over 300 feet in height and inaccessible except by the use of mechanical means of suspension. Eleven of these extend entirely across from top of high bank to top of high bank, a distance of about twelve to fifteen hundred feet. The detailed map, however, while costing something has fully repaid the expenditure, in that by having it better locations have been made at the several sites than were possible before such a map was at hand. For instance, at Dam Site No. 1, the general location selected in 1890 shows a bottom length of dam of about 650 feet and a length, at the flow line of a 130-foot dam, of 914 feet. The new location made after detailed surveys this year shows a bottom length of 500 feet, and a length at the flow line of 790 feet, the top lengths being measured from natural surface to natural surface in each case. At the price of rubble masonry used in the estimates a single linear foot of structure at this site will cost nearly $2,000. Whence it follows that, other conditions remaining the same, the value of the surveys at this site alone is in the vicinity of $300,000.

At Dam Site No. 2, the bottom length of the new location is about the same as the old; but the top length of the new location is 880 feet at the 130-foot line, as against 972 feet in the old. The greatest gain at Site No. 2 has been in demonstrating by the detailed surveys that for a 130-foot dam an independent spillway can be constructed at that location without very materially increasing the cost over what it would be for a dam arranged with reference to spillway over the front, as is required at Site No. 1.

The advantages of an independent spillway are so obvious as to hardly require mentioning, although briefly discussed further on.

The detailed surveys, however, have not been extended to cover the land lines of the submerged area, it being concluded, after discussion of the question with yourself, that the large expenditure required to fully work up the land lines was unnecessary until after the work was actually begun. In any case two or three years will elapse after beginning construction before any area will be actually flooded, and ample time will therefore be afforded for the completion of the land surveys. As regards land immediately in the vicinity of the structure, and which will be required at once for the location of the various works, it is considered that no difficulty will be experienced in making the necessary arrangements.

Another reason for not making the land surveys a part of this year's work was the smallness of the appropriation — there was hardly money enough available to carry on the land surveys, and at the same time complete the thorough examination of the foundations which has been deemed essential in order to prepare reliable estimates of cost.

It may be remarked in passing that much benefit has been derived in the course of the examinations from conversations with the consulting engineer, Hon. E. Sweet. This gentleman visited Mount Morris on Tuesday, October 10, 1893, and spent the time from 11 a. m. to 4 p. m. in viewing the Hogback location, and the tests then in progress at Site No. 2. On November twenty-fourth, Mr. Sweet again visited Mount Morris and looked over Site No. 1, where soundings and borings were at that time in progress. In the afternoon of the same day he examined the cores from a number of bore holes, as preserved in boxes for future reference.

In any structure of the magnitude of the proposed Genesee river storage dam, the adequacy of the foundation is of first importance. An error of judgment as to its sufficiency could easily lead to either largely increased expenditure in the work of construction or even to an entire failure of the project. In order to avoid either contingency such examinations must be made as will exhibit the

character of the underlying strata in all its details. In the present case a compact shale, known in the nomenclature of the State Geological Survey as the Genesee shale, exists ten to fifteen feet below the bed of the river at the several sites examined and from twenty to twenty-five feet below the surface of the adjacent flats. The general character of this shale has been referred to in the preceding reports and no further notice is required here, and we may proceed, therefore to an account of the detailed tests which have actually been made.

By reason of considerable previous experience in borings, the present writer was of the opinion that more definite information could be obtained by diamond drilling than by any other means. This view having met with your approval, drillmen were corresponded with, and after satisfactory arrangements were perfected an outfit got to work on September eighteenth. At the same time the work of determining the top of the rock by soundings and test pits at a large number of points was actively prosecuted. From September eighteenth to September thirtieth the diamond drill was operated ten hours per day, but from October first to November twenty-eighth, when the work was finally completed, the drill was operated twenty hours each day. The following is the detail of the drill work actually accomplished:

At Dam Site No. 2, from September eighteenth to October twenty-first, thirteen holes were drilled, making a total length of 727.7 feet, and an average length of fifty-six feet.

From October twenty-three to November tenth, nine holes were drilled at the Hogback location, making a total length of 563.9 feet, and an average of 62.7 feet.

From November twelve to twenty-eight, seven holes were drilled at Dam Site No. 1 making a total length at that site of 388.1 feet and an average length of 55.4 feet.

The total number of holes drilled at all the sites was therefore twenty-nine, making a grand total length of 1,679.7 feet.

The top of rock was determined by soundings at seventy-eight points at the three locations studied, the total depth of sounding being 1,256 feet and the average depth 16.1 feet.

In addition to the borings and soundings 100 test pits were sunk to a total depth of 1,217 feet, the average depth being 12.2 feet. A number of the test pits were about thirty feet deep.

The cores taken out by the diamond drill give a satisfactory exhibit of the underlying strata and taken in conjunction with the water pressure test, which were applied to nearly all the drill-holes, indicate with clearness the kind and quantity of treatment necessary to make a safe and stable foundation on these shales.

The water pressure tests may be properly referred to a little in detail.

Obviously it is transcendently important to secure an absolutely water-tight foundation for a high dam. In order to accomplish this it is necessary that the underlying strata be either naturally impervious or that they may be made so by some artificial treatment. When not impervious the necessary special treatment may be an important item of the total expense. The securing of definite information as to the perviousness or imperviousness of the strata became, therefore, an important and indeed indispensable part of the preliminary study. To gain this information seed bags set to different depths in the drill holes were first tried. By means of suitable pipe and flexible hose connections, water under from 100 to 200 pounds pressure per square inch was forced down through the bag and the effect observed. The length of time required for the bag to swell, however, made this method rather cumbersome, and an improvement in method was effected by the use of a rubber packer so arranged that it could be packed tight, instantly, by mechanical arrangements. With such a packer and the flexible connections it was very easy to first set the packer at or near the bottom of each drill hole and, applying the pressure from the pump, examine the hole in detail from bottom to top. A pressure gauge indicated at once any loss of water through seams and crevices, the amount of the loss being, of course, proportional to the decrease in pressure. In some cases where considerable loss occurred the put-

ting in the hole of a few quarts of wheat bran was sufficient to make everything tight.

After going over the drill-holes in detail with this test, if any considerable leaks were found, the pump was then ran for several hours and the effect observed. In order to find out where the water went to several pounds of permanganate of potash, an intense coloring material, were added in a number of cases. This test was, however, entirely without result, other than that the coloring matter forced down into one drill hole appeared and as pressure was applied, ran out at another drill hole 100 feet distant. Asides from some indications of this character absolutely no traces of the coloring matter could be obtained. In a number of cases the effect of pumping for several hours was to gradually increase the back pressure against the pump. In such cases after the pump was disconnected, water ran out of the drill hole for from one-half an hour to two hours, a result clearly indicating the presence of vertical seams at these points. As to whether these vertical seams are likely to prove troublesome, it may be said that none of any considerable size have been discovered and in view of the moderate silt-bearing character of the stream when in flood flow it seems probable that such small ones as exist will soon silt up and become entirely water tight. This process may be made a gradual and perfectly safe one by allowing the water to rise for short periods of flood flow a few times each year during the construction. By intelligent management on these lines the whole reservoir may be made practically impervious by the time it is ready for permanent use.

Proceeding on these lines systematic tests of all but one or two of the drill holes were made and the results recorded. Some of the details will be presented in the final report, but for present purposes it is sufficient to say that the net results of the soundings, borings, test pits and water pressure tests indicate that as a whole a high dam can be built at either of the three locations examined without any serious difficulty — the obstacles to be overcome

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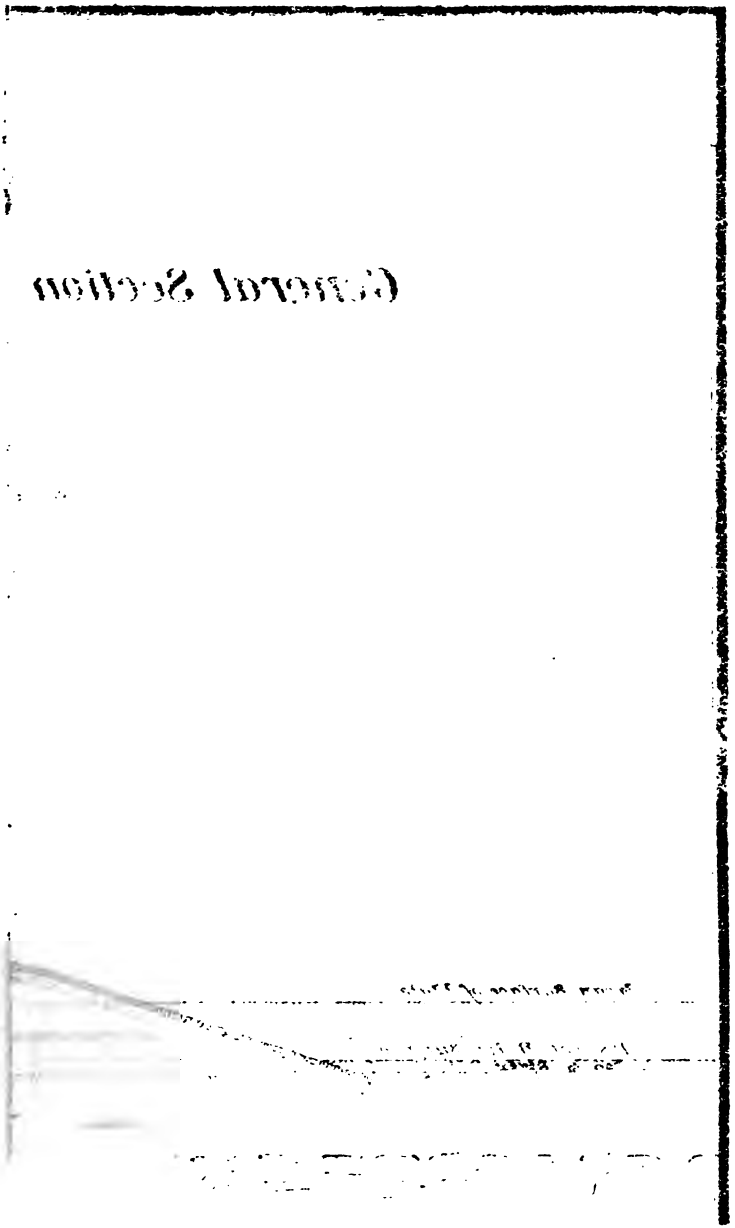
are such as may be expected to yield easily to ordinary engineering expedients. As a whole the quality of the rock in the foundations at Dam Sites Nos. 1 and 2, is somewhat better than at the Hogback, while No. 1 may be considered slightly better than No. 2. At all of the locations it appears desirable to extend a concrete trench about thirty feet in the rock and probably it will be well to drill a number of holes in the bottom of the trench excavated for this purpose into which grout will be pumped under pressure. A similar treatment may also be required under a portion of the main foundation as well as at the ends. It is considered that the item "grouting in foundations," in estimates is ample for this work.

The foregoing discussion of the examinations for foundation indicates that the work has been carried on with reference to a masonry construction. In my judgment no other form of construction should be seriously contemplated, although it is recognized that an inclined plane of wrought iron or steel plates with trestle supports is entirely feasible. Such a construction, while probably a small per cent less expensive than masonry, must be considered as lacking somewhat the permanency of well executed stone work. It is possible, moreover, that an iron spillway might be designed for Dam Site No. 1 of somewhat less expense than the granite masonry construction on which the estimates have been based, although it is exceedingly doubtful if any iron construction could be made entirely permanent for such purpose. For these reasons it has been deemed preferable to base all estimates of cost on masonry construction.

In considering the various forms of dams applicable to the several cases under consideration we are confronted with the fact that the flood flows of the Genesee river are very large, and that therefore exceedingly liberal spillway arrangements must be provided. The spillway, moreover, is an important element of total cost, and it is accordingly important that no greater length of spillway be provided than absolutely necessary. The only direct observation bearing on the case is that cited by Mr. Kibbe in his report in the 1890 annual, where is given an actual observa-

tion on September 11, 1890, of a mean depth on the crest of the old Mount Morris dam of 6.8 feet. The crest on which this observation was taken is 337 feet in length. The citizens of Mount Morris state that the water frequently stands at an elevation of eight feet on the crest of this dam when the river is in flood flow. The depth of eight feet, however, is more than should be admitted on the crests of high dams and for the present the estimates have been based on a total length of spillway of 400 feet. In the final report an attempt will be made to deduce the proper length of spillway from a consideration of the rainfall in comparison with stream flow, though the data bearing on this part of the subject is unnecessarily scanty. In 1890 Mr. Kibbe established a gage at the old Mount Morris dam, and daily observations of the flow of the stream were taken during the few months covered by his studies. It is exceedingly unfortunate that this series of observations was not continued without intermission until the present time, as, had this been done, we would now be, without doubt, in possession of such data as would permit this part of the problem to be solved on its merits. A gage was again established under my direction in August of this year, but since that time the precipitation has been so small that no flood flows have occurred, the highest being 3.39 feet, on August twenty-ninth. This year's observations are, however, valuable as illustrating how small the flow of the stream may at times be for several months. Arrangements have been made for continuing these gagings during the coming winter and spring, and it is hoped that the spring freshets will furnish some more definite information on this exceedingly important part of our subject than any yet at hand.

As bearing on the economical construction of the work, the proper location of the necessary transportation line is a matter of considerable importance. The commission of 1892 has based its recommendation of location at Site No. 1 largely on the assumed fact that a line of railway only 1.2 miles in length can be constructed up the line of the gorge near the water's edge and connecting with the Western New York and Pennsylvania railway just below the river bridge. An examination of the project in



detail indicates, however, that such a line would be undesirable for the following reasons:

(1) The line, if actually laid on the flats near the water's edge, would require two crossings of the river in order to reach Dam Site No. 1, or four crossings to reach Dam Site No. 2, all of which, with a portion of the embankment would be swept away with the spring floods.

(2) If built in side-hill work, thereby avoiding the river crossings, the expense of grading alone would reach about $20,000 for a line to Dam Site No. 1, or about $35,000 to $40,000 for a line to Dam Site No. 2.

(3) A large amount of space is required at the dam terminal for the proper handling of the material. During the working season about fifty cars per day would be required. The sidings for these together with the necessary sheds, hoisting and mixing machinery, etc., would require far more space than exists on the flats.

(4) The location of the railway in the gorge would necessitate the location therein of the sheds, hoisting and mixing machinery, etc., and their consequent exposure to disaster with each year's flood.

(5) Such a location also involves the lifting of all the material above the foundation with the resulting addition to the expense of from five to ten cents per yard over what it will be if the material can all be taken from the cars to the work by gravity.

(6) The total cost of the work can be decreased somewhat by so locating the transportation as to deliver all material on the top of the high bank where it can all be conveyed from the cars to the work by gravity purely.

The amount of material to be transported for a 130-foot masonry dam is roundly 700,000 net tons. The bulk of this is stone which, with the exception of the small amount of granite required, will all naturally come from some point on the line of the Western New York and Pennsylvania railway to the south of Mount Morris. The proper location of the transportation line is therefore from some point on the Western New York and Pennsylvania

railway, south of Mount Morris, to a point on top of high bank just above the selected location for dam. A line of this character has been located from a point on the railway about one and one-half miles above Tuscarora station to the top of high bank at Dam Site No. 2, a distance of seven and one-half miles Should Site No. 1 be selected, this line can be readily extended a distance of 4,700 feet along the top of the high bank to that site.

Lack of time has prevented the preparation of any detailed estimates of cost of transportation lines in time for this report, and complete consideration of this part of the subject is necessarily delayed until some later report. For present purposes I have assumed cost of transportation line on the route indicated to Dam Site No. 2 as $45,000. For Site No. 1, we may assume for comparison, the construction of a line on the west side of the river from some point on the Delaware, Lackawanna and Western railway, about two miles west of the Western New York and Pennsylvania junction to a point on the west high bank at Dam Site No. 1, the cost of which has been taken at $25,000. If extended along the top of the high bank to Dam Site No. 2, such a line would cost about $30,000. It is believed, however, that in case Dam Site No. 1 is selected, the longer line will prove the cheaper when all the elements of transportation are taken into account, for the reason that an approach to this site from the west side of the river involves transportation over two independent lines of railway before the branch line is reached, with a total additional distance to the branch line of fourteen miles. My reasons for stating that the longer line will probably in any case prove the cheaper, appear in detail in the comparisons following. In any case it will be noticed that the foregoing statement is based upon the assumption of bringing all the material, except granite, from some point to the south of Mount Morris. On this point it may be stated that the fine sandstones of northern Pennsylvania offer an opportunity to obtain a durable, easily-worked stone at moderate cost. At many places in the northern part of that State these sandstones exist on the surface of the ground in vast quantities in the form of boulders, frequently of

many hundred cubic yards content. On the line of the Western New York and Pennsylvania railway, near Oil City, in particular, such stone are found in sufficient quantity for our purpose directly on the line of the railway, and reliable parties signify their willingness to load them on cars at about two dollars and fifty cents to two dollars and seventy-five cents per cubic yard. The railway has made a rate to either Tuscarora or Mount Morris of eighty cents per net ton. Transportation over the branch line from above Tuscarora, a distance of either seven and one-half or about eight and one-half miles, depending on which site is selected, will not exceed, aside from first cost of line and maintenance, about five cents per net ton. We have, then, as the net cost per cubic yard of these stone on cars at the work, the following:

Cost of quarrying and loading on cars, say	$2 75
Transportation over Western New York and Pennsylvania, two net ton, at 80 cents.....	1 60
Transportation over branch line, two net tons, at 5 cents,	10
Cost on cars at dam	$4 45

These stone will be used for the face of the dam, gate-house, protection and retaining walls, except in the case of Dam Site No. 1, where granite has been provided for the face work of the spillway.

The backing will be obtained either from Portage or Nunda, where stone of good quality for this purpose are found in quantity. On these stone the Western New York and Pennsylvania railway offers the following rates:

Portage to Tuscarora, thirty-five cents per net ton.

Nunda to Tuscarora, thirty cents per net ton.

Portage to Delaware, Lackawanna and Western junction, forty cents per net ton.

Nunda to Delaware, Lackawanna and Western junction, thirty-five cents per net ton.

Stone of the proper size for backing can be quarried and put on cars at either Portage or Nunda at from one dollar to one dollar

and fifty cents per cubic yard. We have then for the cost per cubic yard of these stone on cars at the work, the branch line from above Tuscarora being assumed, the following:

Cost of quarrying at Portage and putting on cars.....	$1 50
Transportation over Western New York and Pennsylvania, two net tons, at 35 cents	70
Transportation over branch line, two net tons, at 5 cents,	10
	$2 30

From Nunda the same stone will cost two dollars and twenty-five cents per cubic yard on cars at the work.

We may now rapidly compare the proposed branch transportation lines, taking into account the transportation of all the stone required for a 130-foot dam at Dam Site No. 2, except the granite, of which only a small amount will be used at this site.

(1.) The branch line from above Tuscarora to top of high bank at Dam Site No. 2:

Eighty thousand cubic yards of Pennsylvania sandstone, Western New York and Pennsylvania railway charge, at $1.60 per cubic yard	$128,000
Cost of transportating same over branch line, at 10 cents per cubic yard	8,000
Two hundred and forty-seven thousand cubic yards Portage stone, over Western New York and Pennsylvania railway, at 70 cents per cubic yard	172,900
Cost of transporting same over branch line, 10 cents per cubic yard	24,700
Original cost of branch line	45,000
Cost of maintenance for six years	9,000
Amount	$387,600

(2.) The branch line, from point on the Delaware, Lackawanna and Western railway, to top of high bank at Dam Site No. 2:

Eighty thousand cubic yards Pennsylvania sandstone, Western New York and Pennsylvania railway charge, at $1.60	$128,000
Cost of transporting same, short haul, over Delaware, Lackawanna and Western railway, at 20 cents per cubic yard	16,000
Cost of transporting same over branch line, at 10 cents per cubic yard	8,000
Two hundred and forty-seven thousand cubic yards Portage stone, Western New York and Pennsylvania railway charge, at 80 cents per cubic yard	197,600
Cost of transporting same, short haul, over Delaware, Lackawanna and Western, at 20 cents per cubic yard	49,400
Cost of transporting same over branch line, at 10 per cubic yard	24,700
Original cost of branch line	30,000
Cost of maintenance for six years	5,000
Amount	$458,700

It, therefore, appears that there will result an economy of ($458,700 — $387,600) = $71,100 by the construction of the longer line.

(3.) For the case of a line directly up the gorge from a point on the Western New York and Pennsylvania railway near the Genesee river bridge the discussion may take the following form:

As in the cases already discussed it will be assumed that the material be delivered at Dam Site No. 2:

Eighty thousand cubic yards of Pennsylvania sandstone, Western New York and Pennsylvania railway charge, at $1.60 per cubic yard	$128,000
Cost of transporting same over branch line	8,000
Two hundred and forty-seven thousand cubic yards Portage stone, Western New York and Pennsylsylvania railway charge, at 80 cents per cubic yard,	197,600
Cost of transporting same over branch line	24,700

Original cost of line taken at.....	$35,000
Cost of maintenance for six years	7,500
Contingency on sheds and works located in gorge....	15,000
Average additional cost per cubic yard of 150,000 cubic yards of masonry, on account of being mechanically hoisted into place, at 5 cents per net ton — say 10 cents per cubic yard	15,000
Amount	$430,800

In this case the resulting economy would be ($430,800 — $387,600) = $43,200. If Dam Site No. 1 were to be selected the last figure would be reduced by about $20,000 to $25,000, or the resulting economy would be approximately say ($43,200 — $25,000) = $18,200.

In these comparisons the cost of transportation has been taken the same on all the branch lines without reference to length. This has been considered proper in view of constant employment of only one locomotive and train gang in either case. At any rate the advantage of delivery on top of the bank is so great that independent of the clear financial saving the line from the Western New York and Pennsylvania railway above Tuscorara appears superior to any other that can be located. But in making this statement, it will be understood, as assumed, that the material come from Portage, Nunda, Northern Pennsylvania, or some other point to the south of Mount Morris.

The line in question will have grades of between eighty and ninety feet to the line for the first two and a half miles. The balance will not at any place exceed, except for short stretches, about fifty feet.

The proposed line from the Delaware, Lackawanna and Western railway would have about ninety feet to the mile grade for three miles.

The construction of the suggested transportation line will make necessary the temporary occupation of a strip of land seven and one-half or eight and one-half miles in length, depending on the site selected. The most of this is through cultivated farms

and provision should be made for a temporary occupation of about six or seven years.

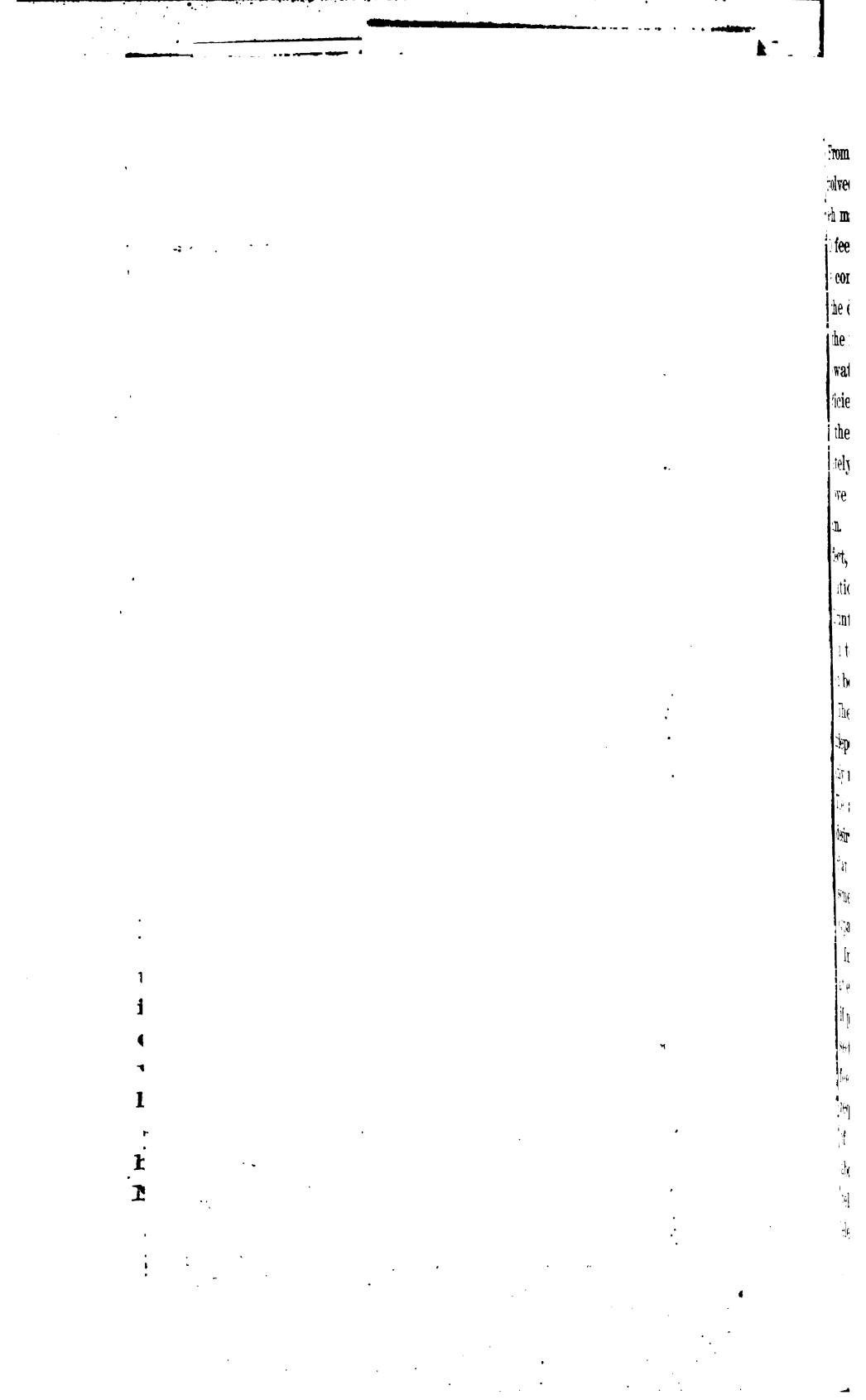
The preceding discussion has indicated the probable source of the bulk of the material to be used in the construction of a masonry dam. In addition to what has already been indicated, it may be stated that there is, at Ridge post-office, about three miles from Dam Site No. 2, a quarry of stone of the same general quality as that at Portage and Nunda. As now opened this quarry shows a depth of workable stone of seven to eight feet, but whether it has much greater depth is as yet unknown. The amount of stone required is so large that any quarry selected should have a working depth of at least from fifteen to twenty feet. It was intended to sink a few diamond drill holes at this quarry but the approach of cold weather before the completion of the balance of the work prevented carrying this intention into effect. This quarry is located about one mile from the proposed transportation line from above Tuscarora to top of high bank, and at least 100 feet above said line and, if future developments indicate that it is available, its utilization is in line with the arrangements here suggested. It is also about 150 feet above the top of the high bank at Dam Site No. 2, and the topography is such as to admit of the location of a surface gravity line to that point. In any case the situation of this quarry is such that some expense for pumping or drainage would necessarily be incurred in order to keep it clear from water. At both Portage and Nunda, however, this difficulty does not exist. At both places quarries may be opened in the steep side hill far above the railway track, and material loaded on cars by gravity, whereas at the Ridge quarry the material would necessarily require hoisting on to the cars. The rate of transportation given is so reasonable that it is considered probable, taking everything into account, that the material can be obtained more cheaply from either Portage or Nunda than from Ridge quarry, even though the haul is considerably longer. For the present, however, the actual decision is left open until a more complete analysis can be made. Whichever of the local quarries may be finally selected, the stone

therefrom should be used for backing only. All face work should be either of the Pennsylvania sandstone or some stone of equal grade, and has been so provided in the estimates.

For the forming of the spillway at Dam Site No. 1, granite masonry in large blocks has been estimated. A small amount of granite is also included for the more exposed portions of sluiceways, etc.

The estimates have all been made and the discussion thus far carried on under the assumption that the dam and retaining walls, etc., be composed of a good class of well-laid rubble masonry. A considerable quantity of concrete has been provided for, to be used for making water-tight work in the foundations and at the sides of the gorge, and also for protection of spillway at Dam Site No. 2. It has also been suggested that the whole structure be constructed of concrete. In view of the possible economy resulting from the use of this material on a large scale, an extensive series of experiments with different available raw materials have been carried out. About 180 cubes of one foot have been molded, and after these have attained some age they will be broken by compression and the results noted. In this way it is hoped that some useful information as to the value of concrete from the available local material may be gained. The stones used in these experiments are the Genesee shale, and stone from Ridge, Nunda and Portage quarries. These have been combined in various proportions with Cumming's Buffalo, Newman's Akron, Bang's and Gaynor's Fayetteville, Norton's Rosendale, Millem's Wayland Portland and Saylor's Portland cements. In each case two series of blocks have been made, one of which have been set in water while the other have been left to season in air. A record of all the attendant conditions has been kept, and all the details will be given after the compressive tests have been made, in a later report.

Sand of proper quality for the work can be obtained within one-half mile of Dam Site No. 1, or within about one mile of Dam Site No. 2.



From an engineering point of view the most difficult question involved in these designs is the arrangement of the spillway in such manner as to destroy the mechanical energy of a waterfall 130 feet in height. In the design of a dam for site No. 1, where the conditions are such as to necessitate spilling over the front of the dam, this is partially accomplished by stepping the masonry in the manner shown on the accompanying section. The quantity of water, however, is so great that this will be itself hardly sufficient. Further assistance in this direction will be obtained by the use of a water cushion, whereby the water surface immediately below the dam will be permanently raised to sixteen feet above the present ordinary water surface of the old Mount Morris dam. It is proposed to locate the subsidiary dam, producing this effect, just above the Mount Morris bridge, the advantage of this location being, the creating of an additional water power at Mount Morris of sixteen feet over and above the present power, or a total head there of thirty-four feet. The water cushion wall can be as cheaply constructed there as at any other point.

The detailed surveys show that for a dam 130 feet in height an independent spillway can be constructed at Dam Site No. 2 with only moderate increase of expense over that of a dam at Site No. 1. The advantages of such a spillway are so great it would appear desirable in case it is concluded to build an all-masonry dam of that height to select Site No. 2, even though the selection entailed somewhat greater first cost, as well as some reduction of storage capacity when compared with Site No. 1.

In regard to the use of a tunnel for discharging the flood flow at either of the proposed sites, it may be stated that such a tunnel if placed at a high elevation, would require to be of very large section — for instance, with its center of discharge only thirty feet below the ordinary water surface of the reservoir, it would require, for a discharge of 25,000 cubic feet per second, a diameter of forty feet. The velocity of flow in such a tunnel would be about twenty linear feet per second, and water flowing at this velocity would be ejected from the mouth of the tunnel at an elevation of 100 feet above the present water surface below the

dam. The erosive action of a cylinder of water forty feet in diameter falling 100 feet would be so great as to render artificial protection of the shale bottom imperatively necessary. An efficient protection of the bottom could probably be made as cheaply and efficiently by the water cushion as by any other method.

Again, in case the tunnel were constructed at such lower level as to admit of 130-feet head a great reduction of diameter would of course, result. The velocity of flow through such a low level tunnel would, however, be over 100 lineal feet per second as against about twenty feet for the assumed high level tunnel. A velocity of efflux of over 100 lineal feet per second of a large cylinder of water would, however, require, in order to counteract its erosive tendency, work of the most expensive character, and it is exceedingly doubtful if any gain in economy would result from its use.

Moreover, any possible tunnel, placed below ordinary water level, would be subject to obstruction from ice, and were this to happen the consequent inevitable overflow at the top would be the source of the greatest danger to the structure.

In regard to an all-masonry dam at Site No. 1, it should be remarked that there may always be a possible element of danger in any structure, however solidly constructed, over the face of which spills a stream of the magnitude of the Genesee when in the freshet stage. An examination of the plans of a large number of high dams indicates that usually such provision of length of spillway has been made as will give much less maximum depth than would occur on the crest of the dam proposed for Site No. 1. In the case of the new Croton dam at Cornell's the drainage area above the dam is 340 square miles; in order to care for the flood flows from this area a spillway 1,020 feet in length has been provided. The cross-section of this spillway is quite similar to that designed for the proposed masonry dam at Site No. 1.

The Croton engineers do not consider it safe to provide for less than 150 cubic feet per second per square mile as the possible maximum flood flow, and while this provision is probably some-

A General View of Dam Site No. 1, Looking Down the River.

what larger than necessary in the case of the Genesee it is still important to note that this extreme provision gives a maximum depth on the spillway of only 6.6 feet. The ordinary Croton river floods do not much exceed one-half of this quantity, and it is improbable that the maximum will occur more than twice in a century. In deciding which project to select, difference in cost can properly take, therefore, only relative prominence. The absolute safety and durability of the structures compared should be given due consideration.

The foregoing illustrations, without exhausting the subject, will serve to show the importance of giving the matter of spillway the most thorough consideration before beginning any construction. They also serve to show the vast economic importance of detailed observations as to the flood flow of the stream.

Reference has been made to the Hogback location, a clear idea of which may be obtained from the detailed topographical map prepared to accompany this report. This location combines the advantage not only of permitting the construction of a spillway entirely independent of the artificial dam, but of constructing the necessary regulating works by tunnel through the Hogback also independent of the artificial dam. The gap to be closed at this location is about 700 feet across the bottom, and it has been considered, in view of the opportunity afforded for independent spillway and regulating works, that an earth structure with concrete core would answer every purpose for the main dam. The preliminary studies, while not very complete, indicate that there would be some economy in such a construction at this location, although an all-masonry dam could not be made here with any economy over the other sites. The Hogback location may, therefore, be considered worthy of some further special consideration. This location has not been considered in the previous reports for the reason that in the absence of detailed surveys the engineers have felt some doubt as to the strength of the Hogback itself to support the pressure of high heads of water. A study of the cross-sections indicates that this particular objection need have no special weight. Moreover, the excavation for the independent

spillway, when properly located, would furnish a large amount of surplus material, which could be usefully employed in strengthening up the weaker sections. With this understanding it may be stated that the Hogback is strong enough to support water pressure up to a height of 130 feet. Beyond that height the natural dam could hardly be made to answer.

In order to test the water-holding qualities of the shale material composing the Hogback, a horizontal hole ninety-five feet in depth was drilled at the foot, and a water pressure test applied. Beyond a depth of forty feet a pressure of from 130 to 150 pounds per square inch was maintained for an hour. On continuing the pumping the pressure gradually dropped in two hours and fifteen minutes to forty pounds, thereby indicating the opening up of a number of small passages through the shale. The minuteness of these passages was demonstrated by forcing in a few quarts of wheat bran, when, on resuming pumping, the gage showed 100 pounds. The conclusion was reached, therefore, the same as at the other locations, that by allowing the reservoir to fill for a few times before being actually put in service, at the period of flood flow, the minute crevices which gradually opened up under a pressure of 150 pounds per square inch of clear water, would gradually close up under a pressure not exceeding sixty pounds of water carrying more or less silt.

The question of how to care for the flood flow of the stream during construction is important, not only from an economic point of view, but as bearing on the form of construction to be adopted. After a study of all the attendant conditions it appears that for an all-masonry dam the best plan will be to first build the foundation above the present river bottom, and at that level start a series of arched openings extending far enough across the bottom of the valley to certainly afford the space required to pass the annual flood flow. These openings will be left open during the entire period of construction, the dam being carried to its full height above the arches before the openings are closed. If considered desirable these openings could be left for a year after completing the upper part of the dam, in that way avoid-

ing all danger of unduly straining the masonry of the upper portion before the mortar has attained some age. At the proper time, and during the low water season, the flow of the stream can be diverted through the permanent regulating sluices, and these arched openings closed one at a time by building in with rubble masonry. This plan takes care of the flood flow at all stages of the work, except while putting in the foundation below the river bed. For this portion there is no way except to turn the stream to one side of the valley while the other side is being put in, and so far as necessary use a flume for conveying the small summer flow across the excavation. If serious freshets come during the progress of this work, there is apparently no way except to allow the foundation trenhces and works to be flooded. While some damage may be done it will be of such a nature that when low water comes again, as it will, in a few days, the damage can be easily repaired. How much of such interruption there will be in the early stages of the work is the one unknown factor of the estimates. A knowledge of the regimen of the stream is, however, reassuring, and leads to the conclusion that if ordinary prudence is used in the preparation of coffer dams and flumes the interruptions from this cause may not prove serious.

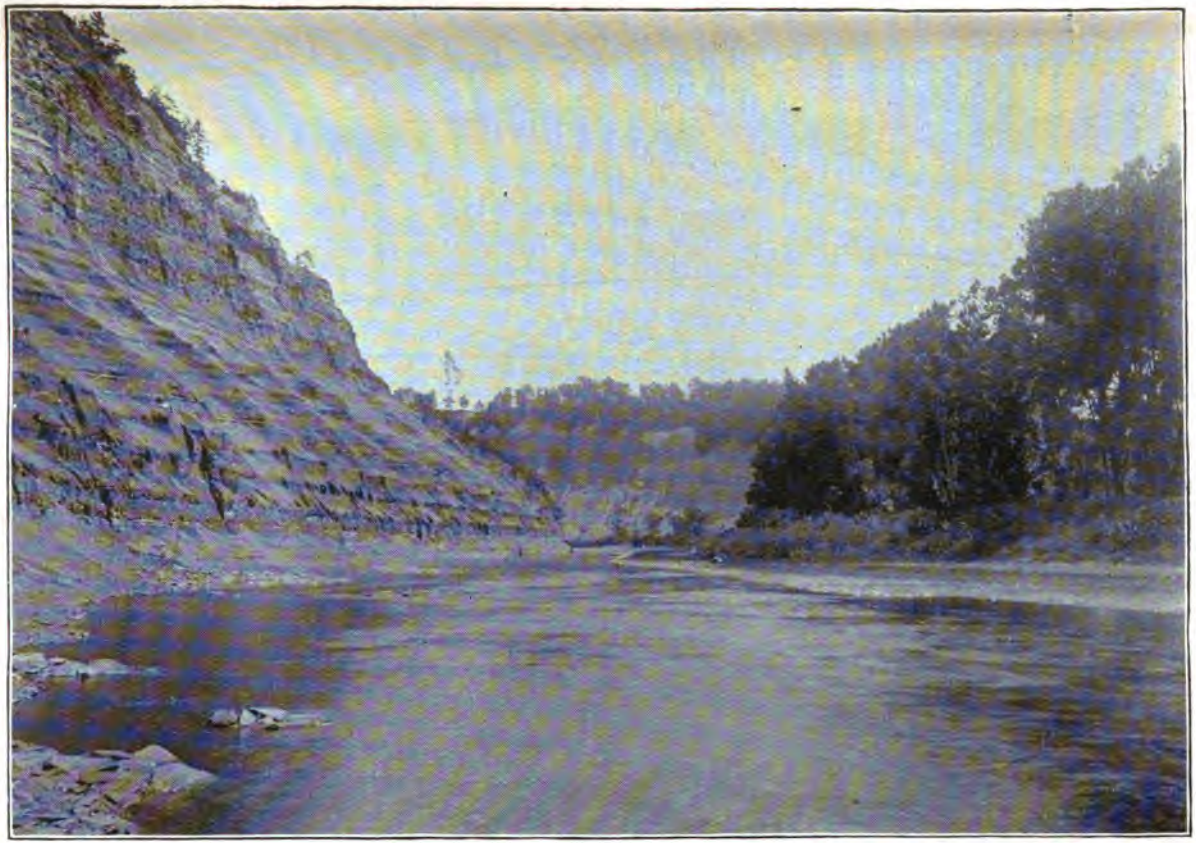
In making the preceding suggestions as to method of caring for flood flow during construction it is recognized that ice may at times prove a source of trouble. To guard against danger of disaster from this cause, it is proposed to leave enough of these openings so that with a small head their discharging capacity is far in excess of the maximum flood flow. With such provision it is improbable to suppose that all would become obstructed at any one time.

The method of caring for the flood flow described in the preceding paragraph is not applicable to an earth dam at the Hogback location where the regulating tunnels through the Hogback would be necessarily utilized for this purpose. The possibility of these tunnels becoming at any time clogged with ice and the consequent partial or complete destruction of the partially completed work

constitutes a practical objection to this location for which thus far no adequate solution has been found.

As to the length of time required to complete the work, it may be stated that after reviewing the work actually accomplished on a large number of high dams it is concluded that for an all-masonry dam the work will necessarily occupy at least five years. The working season will be limited to the seven months from May to November, inclusive, in which there are 180 working days. The total amount of masonry, including the concrete at Dam Site No. 2 is estimated at 327,000 cubic yards. To complete the work in five years will require an average of 65,400 cubic yards per year, or an average of 363 cubic yards for every working day. The working space is so limited it is doubtful if this average could under any circumstances be exceeded. In case the work were too much interrupted by high water during the first two years, the work would probably necessarily extend over six years. Is evident, therefore, that arrangements for occupancy of land, etc., should be made with reference to the foregoing facts. An earth dam at the Hogback location can probably be completed in three years.

The objection to this reservoir has been raised that the Genesee river when in flood flow is a silt-bearing stream and that in consequence any reservoir constructed thereon will rapidly fill up. On this point it may be said that all reservoirs on silt-bearing streams do fill up. So far as casual observation goes, it appears that the Genesee is not bad in this particular, although some deposit, from year to year, may be reasonably expected. The only remedies thus far found efficient are either to construct a silt-dam near the head of the reservoir or to leave large scouring sluices in the dam to be opened each year during the period of flood flow. This latter remedy, however, is only effectual in scouring out the deposited silt in the vicinity of the dam itself. It is entirely without effect on silt already deposited in the upper reaches of the reservoir. In order to be able to use this remedy occasionally it may be desirable to leave a number of the arched openings already described as essential for care of



A General View of Dam Site No. 2, Looking Down the River.

flood flow during construction. These may be closed with timbers at the upper end, and so arranged as to be removable when required. On the other hand, if, in the course of twenty-five to fifty years, the reservoir is found to be silting up to any considerable amount a cheap form of silt dam may be constructed at the upper end. As this fills up in the course of 50 to 100 years more, it may be built higher.

In any case enough has been said to show that the silt difficulty is not a serious one on the Genesee river.

At the present time, in view of the imperative necessity for getting out a preliminary report, before the beginning of the 1894 session of the Legislature, it has not seemed desirable to further delay the work in order to construct sections and prepare comparative estimates for a 100-foot dam. By reason, however, of the strong recommendation by the Commission of 1892, that a 100-foot dam be built; it is recognized that the project of a 100-foot dam should receive some consideration in the present work, and the necessary estimates and comparisons will accordingly be presented in the final report. In the meantime it may be stated that the studies thus far made indicate: (1) That by reason of the necessity for using a water-cushion at either sites Nos. 1 or 2, which will cost the same in either case; and (2) the assumed necessity for an independent spillway, if it can be possibly obtained; and (3) the increased cost of the independent spillway for a 100-foot dam, over that for a 130-foot dam, the actual financial advantage of the 100-foot dam is much less than would appear from a cursory study only. Moreover, the gain in storage between 100 feet and 130 feet is about equal to the gain between fifty-eight and 100 feet. Hence, the construction of the dam to the height of 130 feet, gives about double the storage, over and above that gained by the fifty-eight-foot dam, that will result from the construction of a dam to the height of 100 feet. The cost of a 130-foot dam is not double that of a 100-foot dam. If then the storage has any commercial value, and it can be shown that the additional amount

gained, will be utilized in the immediate or near future, it follows that the 130-foot dam is preferable to the 100-foot.

If, for any reason, it should be deemed desirable to build a fifty-eight-foot dam, without intention of enlarging or building upon in the future, then it is suggested that such a dam be located at either Dam Site 3 or 4, or even some possible site higher up; this policy would leave the lower sites, which are especially available for high dams, storing large quantities of water, available when required for future use.

In view of the probable vast future development of both the transportation and industrial interests of the State, it is the fixed opinion of the present writer that absolutely nothing should be done in the way of a temporizing policy which will, in any way, interfere with the ultimate development of the storage capabilities of this magnificient stream to its approximate full capacity.

The following, are the footings of the several estimates:

At Dam Site No. 1, a masonry dam fifty-eight feet in height, without intention of building or enlarging upon, is estimated to cost $1,200,000.

At the same site a masonry dam, fifty-eight feet in height, but designed of such section as to admit of extension to 130 feet, is estimated to cost $2,234,000.

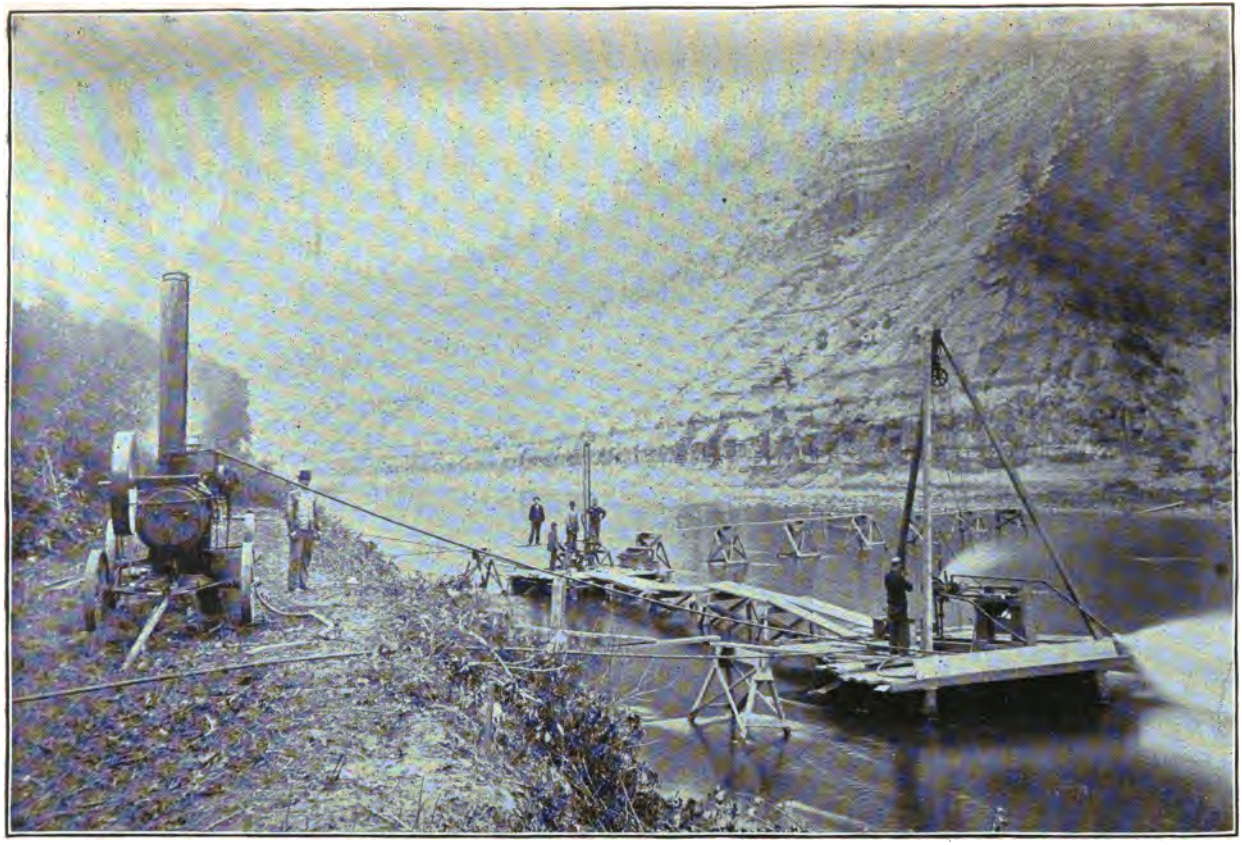
The cost of completing such a masonry dam between the heights of fifty-eight feet and 130 feet, is estimated at $997,000.

The final cost of the completed masonry dam, 130 feet in height, would, therefore, be ($2,234,000 + $997,000) = $3,231,000.

If the 130-foot masonry dam were to be built all at one time — continuously — it is estimated to cost $3,152,000.

The difference between $3,231,000 and $3,152,000, of $79,000, representing the expense of providing temporary protection to spillway, temporary gate-house, cost of removal of same and other miscellaneous items of additional expense of like nature.

The foregoing estimates applying to Dam Site No. 1, provide for a granite-faced spillway 400 feet in length, with water cushion in every case.



Sounding and Drilling Operations at Dam Site No. 2.

Another View at Dam Site No. 2.

At Dam Site No. 2, a masonry dam fifty-eight feet in height, without intention of building or enlarging upon, is estimated to cost $1,269,000.

This estimate includes the expense of a water cushion.

At the same site a masonry dam fifty-eight feet in height, but designed of such section as to admit of extension to 130 feet, is estimated to cost $1,989,000.

There is no practicable way to provide for the flood flow over a fifty-eight-foot masonry dam at this point, except by spillway directly over the front, the same as at Dam Site No. 1. In the case of a dam built to the height of fifty-eight feet temporarily, but designed to be ultimately extended to the height of 130 feet, it is considered that the flow could be allowed to go over the face of the ordinary masonry for a few years, and accordingly granite masonry in front of the dam has not been provided, for the reason that when finally raised to the full height of 130 feet the independent spillway would come into play. The water cushion, however, would be indispensable during the time of flow over the fifty-eight-foot base, and its cost is, therefore, included in the foregoing estimate.

The cost of completing such a masonry dam between the height of fifty-eight feet and 130 feet is estimated at $1,408,000.

The final cost of the completed masonry dam, 130 feet in height, at Dam Site No. 2, would, therefore, be ($1,989,000 + $1,408,000) = $3,397,000.

If the 130-foot masonry dam were to be built all at one time — continuously — it is estimated to cost $3,120,000, without a water cushion, or $3,338,000 with. The reasons for the small increase in total cost, where built piecemeal, are the same as those already cited in the case of Site No. 1.

Owing to the large amount of labor required to complete these several estimates within the time available, the present writer has not been able to give the project of an earth dam at the Hogback other than very cursory attention. In order, however, that the approximate cost of dams at this site might be included in this report for purposes of comparison, Mr. C. L. Johnson, Deputy

State Engineer and Surveyor, kindly undertook to prepare the series of estimates for earth dams, the footings of which herewith appear.

An earth dam fifty-eight feet high at the Hogback location, without intention of building or enlarging upon, with independent spillway and regulating works by tunnel through the Hogback, is estimated to cost $850,000.

An earth dam at the same site 130 feet in height, with independent spillway and regulating works by tunnel through the Hogback, is estimated to cost $2,100,000.

An earth dam at the same site, fifty-eight feet in height, but designed of such section as to admit of extension to 130 feet, is estimated to cost $1,400,000.

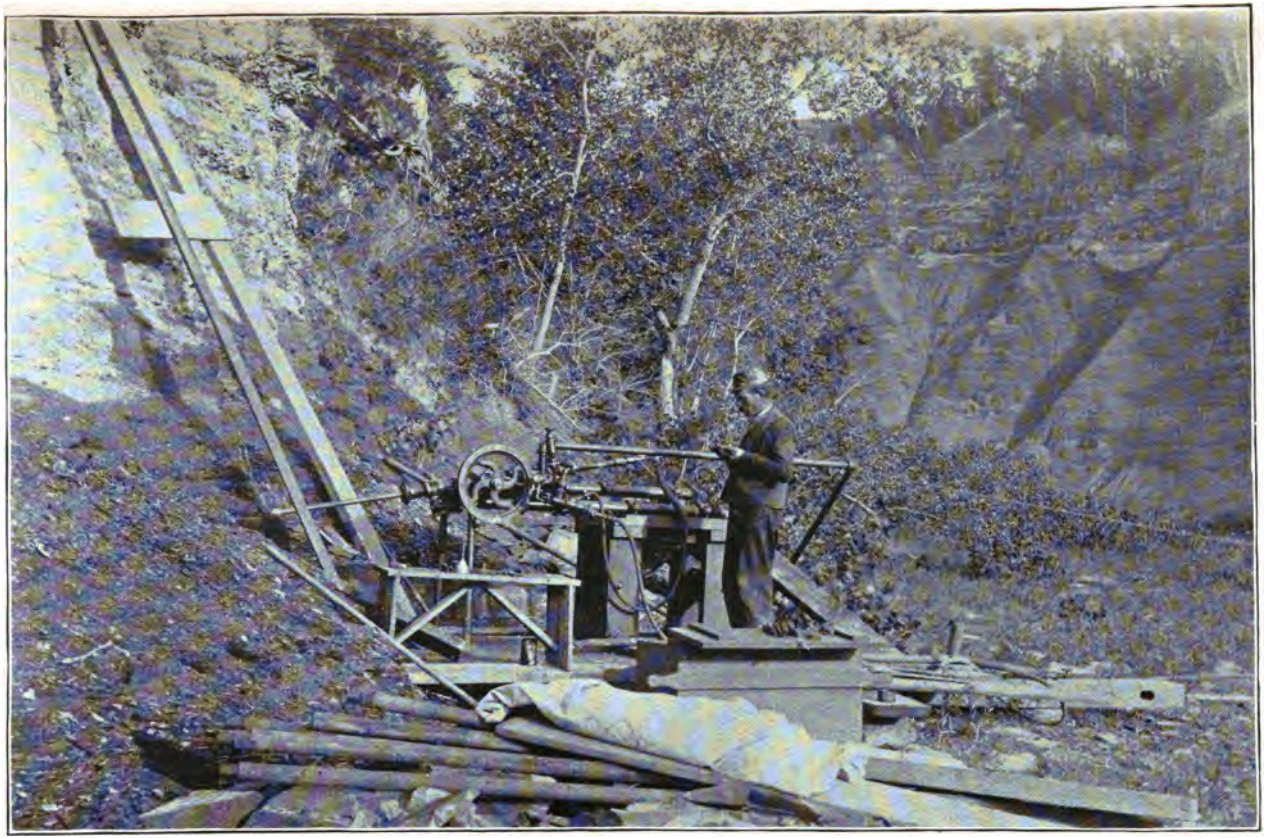
The cost of completing such an earth dam between the heights of fifty-eight feet and 130 feet is estimated to cost $1,050,000.

The final cost of the completed earth dam 130 feet in height at the Hogback location, if built in piecemeal, would therefore be, according to Mr. Johnson's estimate ($1,400,000 + $1,050,00) = $2,450,000.

In all of these earth dams a concrete core carried to a depth in the rock of thirty and thirty-five feet, respectively, is provided for.

In regard to the foregoing estimates, it must be understood that the sections on which they are based, as well as the estimates themselves, have been mostly prepared during the last fifteen days. During the progress of the field work the amount of detail of the various examinations made was so great as to leave little time for close work on plans and estimates. It is needless to remark that a complete report on a project of this magnitude can not be made in fifteen days. At least ninety days' additional time will be required, at the end of which period a final report more nearly worthy of the project can probably be submitted. In the meantime it will be understood that all conclusions and estimates herein contained are subject to a final revision. The detail of the estimates will be given in the final report.

Again, the various sections of dams applicable to the several sites herewith submitted, may also be considered, in some particu-



Diamond Drill Sinking Horizontal Hole at Dam Site No. 2.

lars, as tentative only. This remark especially applies to the top finish, where in the final designs some form of inexpensive architectural ornamentation may be properly applied. At the same time it may be stated that the substantial copings with which the walls are surmounted in the present sections are in every particular a proper finish for an engineering structure of this magnitude; but, if in the preparation of final plans, it should be deemed proper to add any purely architectural features, such decoration could be added without any material increase of cost over that required for the substantial coping alone.

The other possible slight modifications of the sections will not be of such a nature as to materially affect the cost.

Very respectfully.

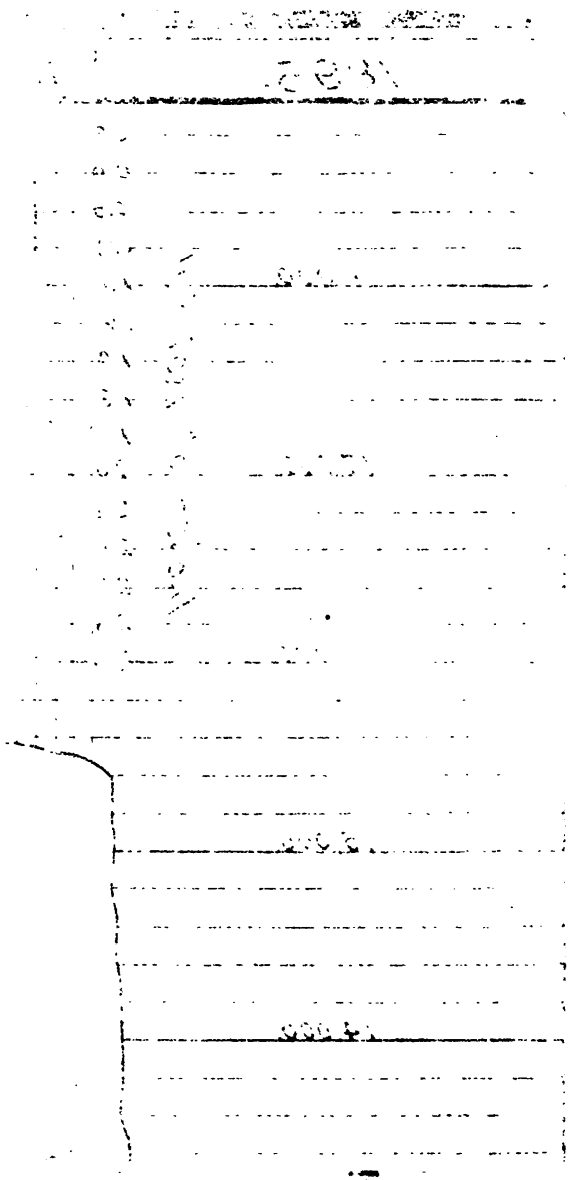
GEO. W. RAFTER,
Engineer in Charge.

FINAL REPORT

ON

GENESEE RIVER STORAGE.

By GEORGE W. RAFTER, *Engineer in Charge.*



REPORT.

ALBANY, N. Y., *April* 1, 1893.

Hon. CAMPBELL W. ADAMS, *State Engineer and Surveyor:*

Sir.—I have the honor to submit herewith my final report on the Genesee river storage project.

In a preliminary report made to your predecessor, the Hon. Martin Schenck, under date of December 15, 1893, it is stated that in view of the suggestion that the proposed storage dam might be economically constructed of concrete from available local material, an extensive series of experiments on this material had been carried out. The experimental blocks have now been tested and the results compiled, and we may properly begin by an exhibit of the information acquired and the conclusions arrived at.

The objects to be obtained by these concrete tests were as follows:

(1.) To determine whether the Genesee shales, which exist in vast quantity in the sides of the gorge and immediately at all the dam sites, could be used as the aggregate of concrete, either for the entire work or for a portion of it.

(2.) To also determine the value of stone from deposits some distance away from the work, but still near enough to be fairly available, for the same purpose.

(3.) To determine the constructive value of such American cements as from their nearness to Mount Morris may be considered especially available for this work.

(4.) To determine the value of the local sand, in combination with the available cements and concrete aggregates.

The stones used in the tests are (1) the Genesee shale, as taken from the sides of the gorge immediately at Dam Site No. 2; (2) one from Ridge quarry, which is located about three miles from Dam Site No. 2, the available information with regard to which

has been given in the preliminary report; (3) from Portage, and (4) from Nunda. The controlling circumstances at these two latter quarries are also indicated in the preliminary report.

The cements used in the tests were (1) Cumming's Buffalo; (2) Newman's Akron; (3) Bangs & Gaynor's Fayetteville; (4) Norton's Rosendale; (5) Millem's Wayland Portland and (6) Saylor's Portland.

The sand used was from extensive banks in the immediate vicinity of Dam Site No. 1.

These several stones have all been broken to pass through a two-inch ring, washed clean and combined with various proportions of sand and cement. One hundred and seventy-four test cubes of one foot each were molded, and a record kept of all the attendant conditions from the time of the moulding until the breaking of the blocks in the month of February. Table No. 2 presents the complete data of the tests.

As preliminary to the tests the voids of the several varieties of stone to be used as aggregates were determined in the following manner: A water-tight cubic foot measure was first filled with the material of which the voids were to be determined. After striking off and weighing, the voids were filled with water, and the whole again weighed. The difference of the two weights represents the weight of the water added to fill the voids of a cubic foot. The ratio of the weight of a cubic foot of water to that of the amount required to fill the voids gives the percentage of the voids.

Proceeding in this way several determinations were made in each case and the means taken. The following are the results reached:

(1) For the Genesee shale, when thrown into the measure with a shovel and slightly shaken, so that the solidity of the material fairly represented that of ordinary measuring in quantity, the mean of five determinations gave the voids as 38.3 per cent. The range of the five was from 36.1 as a minimum to 41.3 per cent as a maximum.

(2) With the shale packed in the measure with a tamping iron, used about as forcibly as for the ordinary ramming of concrete,

the mean of four trials was 31.6 per cent, the range of the four being from 28.8 to 33.6 per cent.

(3) The mean of both of the preceding series (mean of nine determinations) was 34.9 per cent.

(4) For broken stone from the Ridge quarry, slightly shaken, the mean of five trials was 46.2 per cent. The range was from 44.5 to 48.5.

(5) Ridge quarry stone packed gave a mean percentage of voids from five trials of 38.6 per cent. The range was from 36.9 to 39.3 per cent.

(6) The mean of ten trials included in (4) and (5) was 42.4 per cent.

(7) For broken stone from Portage, slightly shaken, the mean of five trials was 43.3 per cent. The range was from 41.7 to 45.3 per cent.

(8) Portage stone packed gave a mean from five trials of 37.4 per cent. The range was from 35.7 to 39.3 per cent.

The mean of the ten included in (7) and (8) was 40.4 per cent.

(10)For broken stone from Nunda, slightly shaken, the mean of five trials was 43.2 per cent. The range was from 41.7 to 44.9.

(11) Nunda stone packed gave a mean from five trials of 38.5 per cent. The range was from 37.3 to 39.7 per cent.

(12) The mean of the ten tests included in (10) and (11) was 40.8 per cent.

The voids in the sand used in the tests were found as the mean of 12 determinations, to be 31.7 per cent. The range was from 35.6 per cent for dry sand from top of pile slightly shaken, to 28.0 per cent for damp sand from the center of large pile packed.

As this sand would ordinarily be measured into work the voids may be expected to average about 32 to 33 per cent.

With the exception of Norton's Rosendale, tensile tests of the several cements used in the concrete blocks were made at the State testing office, and the results are presented in Table No. 1.

Norton's Rosendale was not tested for tensile strength, etc., because the samples forwarded to the State testing office were accidentally destroyed, and the press of work at that office prevented the obtaining and testing of further samples. The con-

crete tests indicate, however, that this cement, when used in concrete, will not vary essentially from the other natural cements of which the detailed tensile and other tests are given in Table No. 1.

In the tensile tests of Table No. 1, a series have been made with the Mt. Morris sand, as well as with the standard quartz used ordinarily for test purposes. Some of the results brought out are of considerable interest, as, for instance, with Saylor's Portland cement, the standard quartz, 1 to 1, gives 329 pounds tensile in seven days and 450 pounds in twenty-eight days, while with Mt. Morris sand, 1 to 1, Saylor's Portland gives only 257 pounds in seven days, and 317 pounds in twenty-eight days. In 2 to 1 proportion the quartz is still superior to the Mt. Morris sand. In 3 to 1 they are about equal; in 4 to 1 the Mt. Morris sand is somewhat superior to the quartz, while in 5 to 1 proportion the sand is still better than the quartz.

If we analyze the tests of Wayland Portland in the same way we observe that for the 1 to 1 proportion the superiority of the quartz over the Mt. Morris sand is much more marked than in the previous case. Thus the quartz gives 436 and 516 pounds in seven and twenty-eight days, respectively, while the sand, 1 to 1, is only 363 and 442 pounds for seven and twenty-eight days, respectively.

At 3 to 1, 4 to 1 and 5 to 1, the quartz and sand are practically equal, although there is a slight superiority of the sand in the 5 to 1 mixture.

An explanation of these interesting results is offered when we examine the column of fineness in Table No. 1, from which it is seen that a sieve of 2,500 meshes to the square inch passes 100 per cent of Saylor's Portland and one of 10,000 meshes passes 97 1-2 per cent of this cement. The Wayland Portland is considerably coarser, 98 3-4 per cent passing the screen of 2,500 meshes, while only 87 1-2 per cent passes the screen of 10,000 meshes.

As between the standard quartz and the Mt. Morris sand the quartz is considerably coarser, the result being that when small

proportions of cement are used as in 4 to 1 and 5 to 1 mixtures the finer Saylor's Portland is able to more thoroughly coat the particles, large and small, of the Mt. Morris sand, than of the coarse quartz. In the case of the Wayland Portland, with 12 1-2 per cent larger than the 10,000 to the inch mesh, we may apparently assume that the 12 1-2 per cent of the cement which fails to pass the sieve of 10,000 meshes is just what is required to fill the voids in the coarse quartz, and hence there results greater homogeneity of the whole mass, with its corresponding increase in cohesive strength. On the other hand, in the case of the Mt. Morris sand, which contains a certain per cent of particles of approximately the same size as these coarse cement particles, there are no void spaces for the cement particles to specially fit into, but they merely act to increase the bulk of the sand; hence there results less homogeneity of the mass, with a corresponding decrease in cohesive strength. It is clear, then, that finer grinding of the Wayland cement would considerably increase its strength when used with the Mt. Morris sand, while with the standard quartz finer grinding would not be likely to lead to any increase in strength and might even lead to a small decrease.

It may be mentioned, however, that even with the disadvantage pointed out, the intrinsic strength of the Wayland Portland is so far in excess of the Saylor's as to still give a 5 to 1 mixture somewhat stronger.

The following is the mechanical analysis of the Mt. Morris sand as made at the State testing office:

	Per cent.
Retained by No. 20 sieve (400 meshes)	0.625
Retained by No. 30 sieve (900 meshes)	7.500
Retained by No. 50 sieve (2,500 meshes)	54.375
Retained by No. 74 sieve (5,746 meshes)	30.625
Retained by No. 100 sieve (10,000 meshes)	3.125
	96.250
Passed by No. 100 sieve	3.750
Total	100.000

Taking the percentages in the order of the amount passed by each sieve and we have:

	Per cent of the whole.
Passed by No. 20 sieve	99.375
Passed by No. 30 sieve	91.875
Passed by No. 50 sieve	37.500
Passed by No. 74 sieve	6.875
Passed by No. 100 sieve	3.750

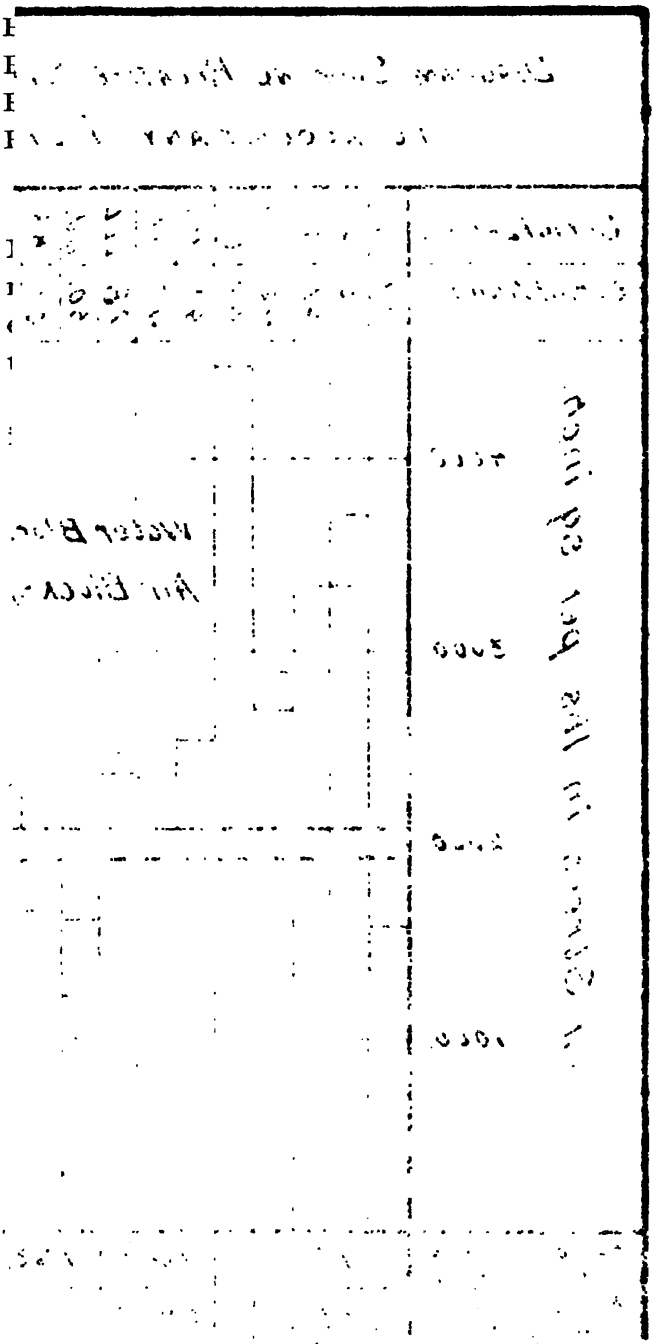
This sand is absolutely pure quartz. Even the portion passing the No. 100 sieve shows clean and sharp when examined under the magnifier. Its entire freedom from vegetable matter, with its other good qualities, makes it an exceedingly valuable addition to the local resources available for this work.

The Wayland Portland neat gives a tensile strength of 542 pounds in seven days and 623 pounds in 28 days. As shown by the wire tests this cement is moderately slow setting — slow enough to give time for any necessary manipulation and handling of material, such as is required on any work of magnitude. The Saylor's Portland, on the other hand, is shown to be nearly as quick setting as the natural cements, a fact which might militate somewhat against its use on a large work like the proposed Genesee storage dam, where, on account of the distances to be covered, some little time for manipulation is imperatively necessary. But in making a final decision the fact should not be lost sight of that when mixed with sand into mortar the time of setting is greatly retarded.

For reasons which appear in detail in the tables, I do not consider any of the natural cements tested available for a work of this character, and I will, therefore, forbear spending any time discussing the results of the tests of these cements.

In addition to the tensile tests just referred to, the cements were also subjected to the hot test, as described by Mr. Faija, in the Transactions of the American Society of Civil Engineers, vol. XVII, page 218, and following:

Both of the Portland cements stand this test perfectly, while the results indicate that none of the natural cements tested would



be satisfactory for important dam work, such as we are now specially considering.

It has been generally assumed that comparative concrete tests should agree in age as nearly as possible to the day, and, accordingly, it may be urged against these tests that the different blocks are not strictly comparable, by reason of varying somewhat in age at the time of breaking. In order to show in this connection just how much importance attaches to exact agreements of age Table No. 1 A has been prepared from the records of the State cement testing office; and while this table shows nothing which was not well known before, it still serves to present saliently in this place that fact that the more rapid increase in the strength of mortars is mostly confined to the first thirty days, and that between ages of two and three months the difference is not so great as to make comparisons entirely impossible. There is, however, a clear improvement with age at this period, and the results of this table will enable one in comparing the concrete tests to make the proper allowance. The Wayland Portland tests, recorded in Table No. 1 A, are not from the same sample as that used for the concrete tests, the tensile tests of which for seven and twenty-eight days' periods are given in Table No. 1.

The manufacture of high grade Portland cements has recently become of considerable importance in this country, and the record of a test by days, up to thirty, of the Empire Portland, is, therefore, also given in Table No. 1 A, by way of showing that there are now a number of high-grade American Portland cements, any one of which may be considered suitable for a work of this character.

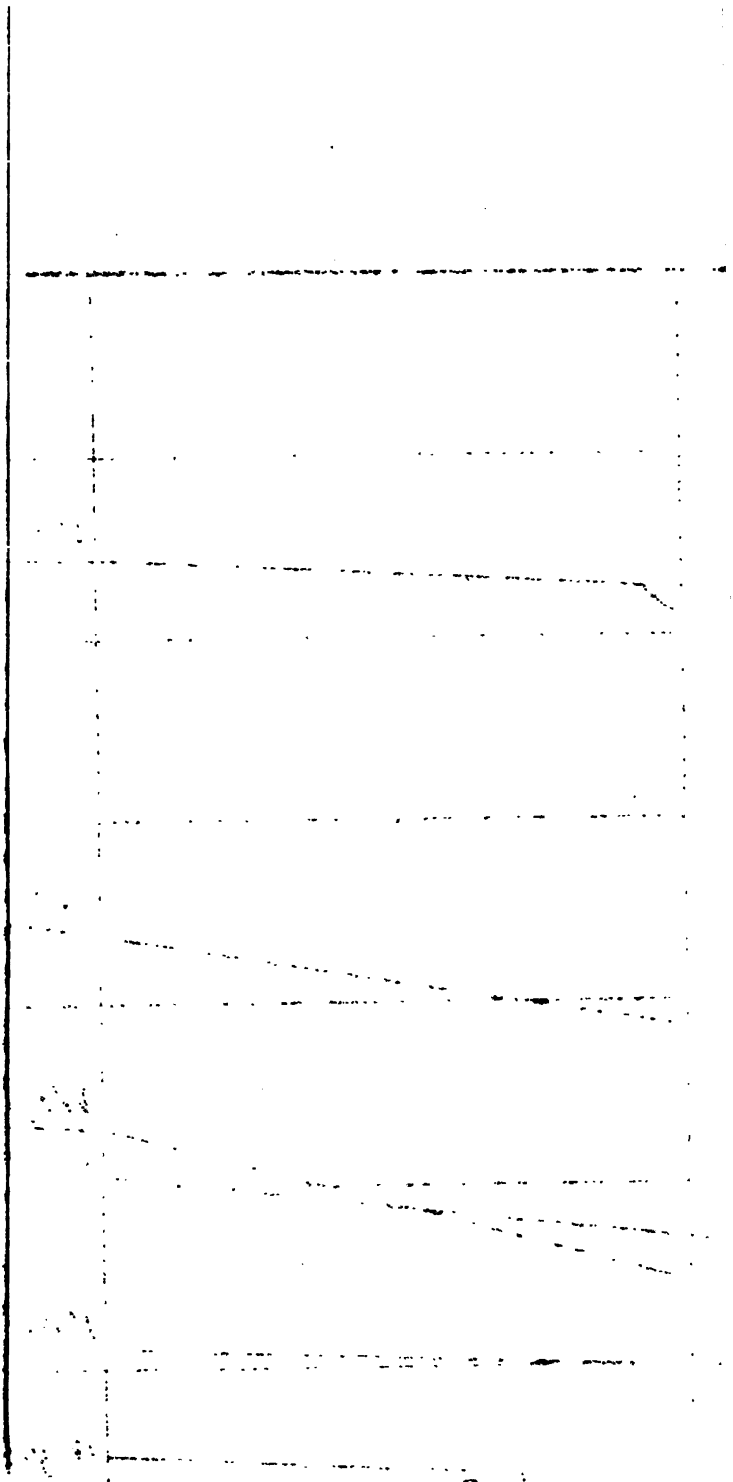
One point of interest brought out by the tests recorded in Table No. 1 A, in comparison with these in Table No. 1, is as to the effect of a slow setting cement on the strength of mortars when mixed with various proportions of sand. On examining the wire test records of the Wayland Portland, given in Table No. 1 A, it will be seen that this sample was considerably slower in setting than that of which the tests are recorded in Table No. 1. Com-

paring the records of the two samples tested neat at seven days and twenty-eight, it appears that the sample of Table No. 1 gave a tensile strength of 542 pounds in seven days, and 623 pounds in twenty-eight days. The sample of Table No. 1 A gave 519 pounds in seven days, and 610 pounds in twenty-eight days, results somewhat less, as they should be, than those of the quicker setting sample of Table No. 1.

If we compare in the same way the results of the tests of these two samples of Wayland cement mixed with various proportions of standard quartz, as given in the tables, we find that the effect of the slower setting cement in retarding the development of strength is much more marked than in the case of the samples tested neat. In 1 to 1 mortar the sample of Table No. 1 gave 436 pounds in seven days and 516 in twenty-eight days, while the slower setting sample of Table No. 1 A gave only 272 pounds in seven days and 386 pounds in twenty-eight days, and not until the end of five months does this sample give approximately the same strength as the quicker sample of Table No. 1 gave in twenty-eight days.

In the same way the two sets of tests may be compared up to the 5 to 1 mixtures, when it will be found that all exhibit the truth of the same general law. In the case of the 5 to 1 mixture the sample of Table No. 1 developed a tensile strength of 152 pounds in twenty-eight days, while the corresponding sample of Table No. 1 A gave a strength of 151 pounds in two months.

In taking the weights of the cements experimented upon the cubic foot has been used as the unit, instead of the bushel, as is ordinarily the case. The weights per cubic foot are as follows: Buffalo Cement Company's cement, 75.5 pounds; Newman's Akron, 75.5 pounds; Bangs & Gaynor's Fayetteville, 75.25 pounds; Norton's Rosendale, 76.0 pounds; Saylor's Portland, 102.65 pounds, and Wayland Portland, 109.25 pounds. These figures are the means of five trials, and represent the weight of the struck measure of cement in each case as shoveled from the barrel into the measure and slightly shaken. The computed weights of the struck Win-



chester bushel are, for Saylor's Portland, 127.7 pounds, and for the Wayland Portland, 135.9 pounds.

These figures show at once that the Wayland Portland is a very heavy cement, comparing favorably in weight with the best foreign brands.

The proportions of water shown in Table No. 1, as used in gaging the cements for the tensile tests, are such as are ordinarily used at the State cement testing office for these cements. As to whether these are the best proportions for these particular cements the present writer is unable to say, as this point has not been experimented upon in the work done under his direction.

In the concrete tests the rule was to only use water enough to produce a stiff paste, except in a few cases, as noted, where the smallest possible quantity of water was used.

From Table No. 2 it appears that the trial blocks of concrete were molded from November 11, 1893, to December 12, inclusive. In every case an attempt was made to mix a large enough batch of concrete to represent the results of actual mixing in practice. To secure this result full measures were in every case taken, thus for a 1 to 3 mortar, 1 cubic foot of cement and 3 cubic feet of sand were used. If amounts of sand and cement were to be combined with 5 parts of broken stone, then 5 cubic feet of stone were measured out.

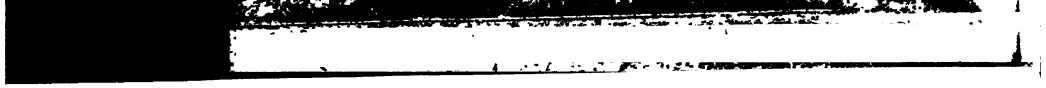
In regard to proportions of sand and cement and mortar, it may be remarked that it appeared to the present writer that the ordinary method of rating concrete as 1, 2 and 4, or 1, 3 and 5, or 1, 4 and 6, etc., meaning thereby that 1 part of cement and 2, 3 or 4 parts of sand, and 4, 5 or 6 parts of broken stone have been used, is not only unscientific, but that it is positively misleading, and its nearly universal prevalence has served to obscure, to a considerable extent the real points to be looked out for in the fabrication of concrete. In reality the relation is between the voids in the broken stone and the mortar.

In pursuance of this view the proportion of mortar to broken stone has been given in the records of these tests as a ratio, the

quantity appearing in column (9) of Table No. 2 being the quotient arising from dividing the number of cubic feet of mortar, measured after mixing, divided by the number of cubic feet of broken stone, with which the mortar was incorporated. The quantity in column (9) expresses, therefore, the percentage relation between the mortar and the stone. The relation which the quantity of mortar used bears to the voids is shown by the statement of results of determination of voids given on a preceding page.

A common rule is that the volume of mortar used should be somewhat in excess of the volume of the voids as measured before compacting. The present tests, however, indicate that the strongest concretes are secured when the volume of the mortar is very little, if any, in excess of the voids as measured before compacting or even a little less. This is strongly illustrated in the series from 39 to EE, in Table No. 2. But in making such concrete, special attention must be paid to the ramming, as upon its thoroughness will depend the success of the operation. In the case of using mortar in less quantity than the volume of the voids, obviously the ramming must be depended upon to reduce, by consolidation of the mass, the volume of the voids to somewhat less than what they would be with a smaller amount of ramming.

As will be seen by inspecting Table No. 2, in the majority of the tests, the mortar was made 42 per cent of the broken stone, this being about the per cent of voids in the hard stones before consolidation as per the determinations already cited. For the Genesee shales this proportion is somewhat too large, but it was considered best to use a relatively larger proportion of mortar with this aggregate than with the others, because by reason of its extreme friableness this stone grinds up considerably in the process of ramming. The comminuted shale acts to dilute the cement and weaken it. This result is strikingly exhibited in all the tests where it appears that the hard stone concretes are from one and one-half to twice as strong as the corresponding shale concretes with the same percentage of mortar. This is shown in detail in Table No. 4. The conclusion is then, that, if mortar



were used in proportion to the smaller volume of voids in the shales the same as with the hard stones, the additional ramming required would so far grind up the shale, thereby reducing the strength of the cement by dilution as to negative the benefits derived from such treatment as applied to hard stone.

If we examine the series of Wayland Portland blocks with 33 per cent of mortar from No. 39 to EE, inclusive, in comparison with the series of 42 per cent of mortar from No. 46 to OO, inclusive, we see, at once, the beneficial effect of decreased amounts of mortar and thorough ramming. In the first series the mortar was only flushed to the surface with the most thorough ramming, while in the latter the flushing occurred after a moderate amount of ramming. Again, in blocks Nos. 46, 47, FF, and GG, the mortar was mixed with only the smallest amount of water possible to be used. The effect of this treatment on the shale blocks was to produce for the shale, with 1 to 6 mortar, in the proportions of 42 per cent to the measured quantity of broken stone, a concrete over 12 per cent stronger than the same strength of mortar in proportion of 33 per cent, as exhibited by blocks 44 and DD. The mean of the final stress of 44 and DD is $\frac{695 + 482}{2} = 589$ pounds, while for 46 and FF it is $\frac{672 + 650}{2} = 671$ pounds, or the mean of 46 and FF is over 12 per cent more than the mean of 44 and DD. On the hard stone blocks, 47 and GG, the effect of the reduction of the amount of water is very much more marked. We have for these two blocks, composed 1 to 6 mortar, 42 per cent of the broken stone, a mean final stress of $\frac{2,174 + 1,882}{2} = 2,028$ pounds on the square inch. This must be considered a very fine result for a concrete with this strength of mortar, and only 76 days old.

As to whether such a result can be obtained in practice on work it may be stated that the use of 1 to 6 mortars will of necessity require very thorough mixing of the sand and cement, but there need be no difficulty in securing this by the use of mechanical

mixers. With this end attained, and through inspection of the gaging of the amount of water to varying temperatures and humidities, and thorough ramming, there ought to be no difficulty in reaching these results always. It must be promised, however, that the inspection be that of an expert in concrete fabrication.

The test blocks were prepared under the supervision of the present writer, who, although unable to be constantly present from beginning to end, was still on the ground directing operations most of the time for the first week, and after that for the next ten days, three or four times each day, a half an hour at a time. At the end of this period the work was fully organized, and for the balance was left in charge of an intelligent assistant. Of the two men actually doing the work of manipulation, the principal one has had long experience in the use of Portland cements and may be considered, as ordinary masons go, an expert at such work. As indicated by Table No. 2, the tests blocks were made late in the fall, after the beginning of freezing weather, and in order to secure as nearly as possible uniformity of conditions, the water used in mixing was warmed to a temperature of 60 degrees. The mixing was according to the following formula, which was rigidly followed in every case. The proportion of cement, sand and stone having been determined upon beforehand, the stone was first measured out by itself on a separate platform and thoroughly washed clean of all impurities by water at a temperature of about 100 degrees. It was fully recognized, however, that nothing like heating the material should be permitted, and care was taken to so proportion the amount of warm water to varying air temperatures as to leave the stone at a temperature not higher than about 60 degrees when actually mixed with the mortar. In order to insure definite results in all matters relating to temperature, a thermometer was used at every stage of the operation.

After washing the stone, the sand was measured on to another platform, evenly spread, and finally the cement measured and also spread evenly over the sand. The mixture was then worked over and over until the cement was evenly distributed throughout the sand, the whole having taken such modified uniform color as was

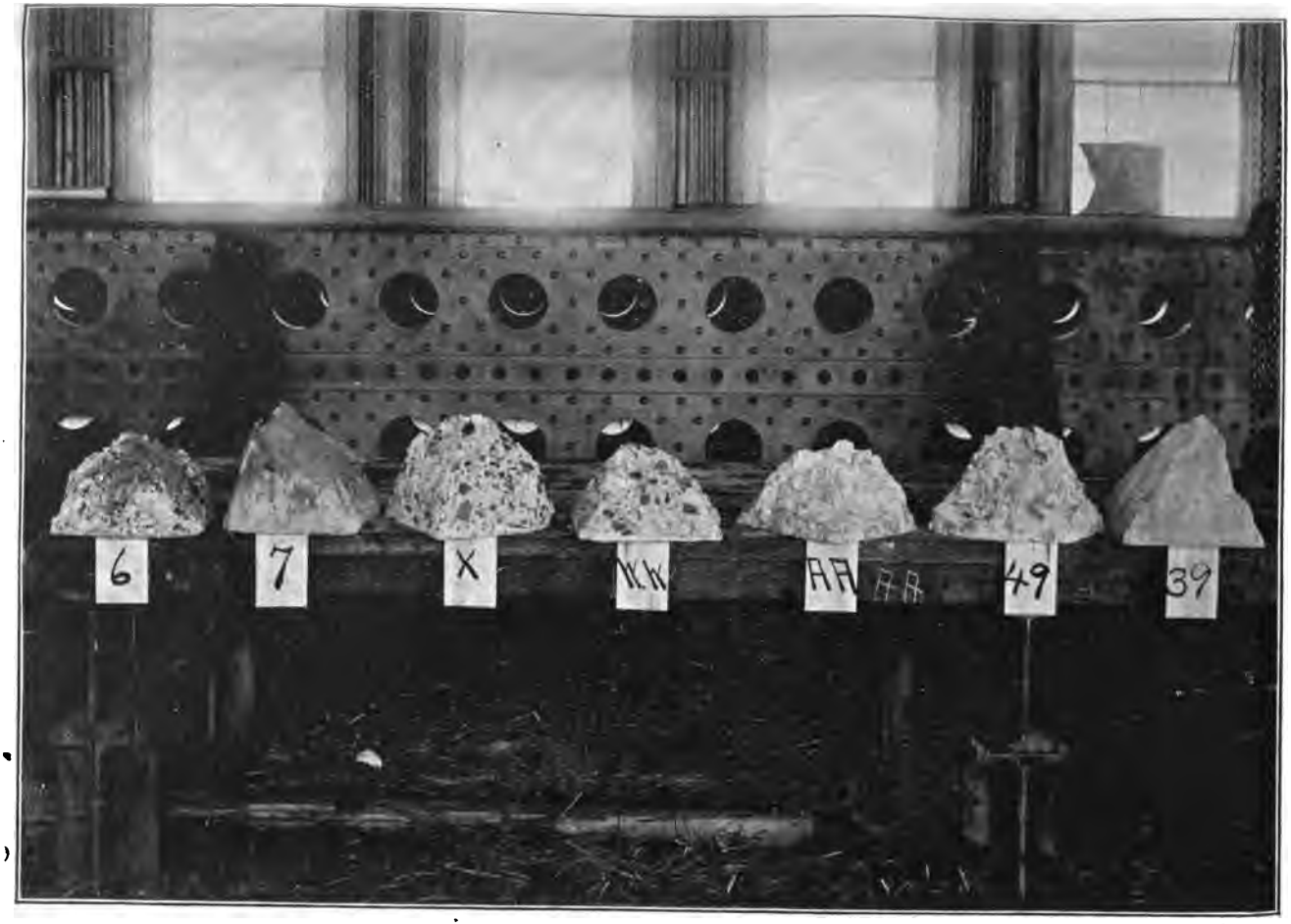


Plate II.— Illustrating Pyramidal Fracture of Concrete Blocks.

due to the varying colors of the different cements. The mixing was continued until absolutely no traces of streaks of cement through the sand could be seen. This operation being completed, enough water was added by sprinkling to bring the mortar to a stiff plastic mass. After thorough incorporation of the water the quantity of mortar to be used was measured from the pile and spread evenly over the broken stone, which had been previously leveled over the platform to the depth of 4 to 5 inches. The mixture of broken stone and mortar was then turned back and forth in accordance with General Gilmore's formula for mixing concrete, as given in his Limes, Cements and Mortars.

The molds used were constructed of thoroughly seasoned two-inch oak plank, and so arranged to go together with wedges as to be easily removed after the blocks had set. In order to prevent the concrete from sticking to the sides of the molds, they were well smeared with tallow before use. The molds having been got ready beforehand, the concrete was shoveled into them and rammed in layers of about three inches, as soon as the process of mixing on the platform was complete. After the first few days the blocks were in all cases made in duplicate, one block of each set going into water at a temperature of 60 degrees, and the other remaining in air at that temperature. In the tables the numbers (1, 2, 3, etc.) indicate the blocks set in water, and the letters (A, B, C, etc.) the blocks left in air. Up to and including the molding of the blocks the operations were all carried on in the open air, and the temperatures given in column (3) of Table No. 2 are the outside air temperatures at the time of mixing each block. As soon as mixed the blocks were carried into a basement, kept at the uniform temperature of 60 degrees, and allowed to stand until set. After setting had taken place and before removing the molds the top of each block was dressed true and smooth with mortar of the same cement and proportion of sand as used in the block itself. The sides of the molds were then removed, but the bottoms were left under the blocks; and after a further interval, depending upon rapidity of setting, those intended for the water test were placed in a tank, where they were entirely submerged.

For both water and air blocks the blocks were left undisturbed on the bottoms until they were finally shipped for testing, January 26, 1894.

The compression tests, of which the details are given in Table No. 2, were made on the large horizontal 1,000-ton machine of the Phoenix Iron Company, at Phoenixville, Penn. For compression tests of this character the machine is arranged with two planed iron faces, one of which is fixed to the machine exactly at right angles to the line of motion, while the other face is free to move in a knuckle joint, which allows it to adjust itself to any slight lack of parallelism of the faces of the blocks to be compressed. In such tests it has been common to make the compression faces true by the use of plaster of Paris, but lacking facilities for applying the plaster of Paris on a plane metallic surface, as recommended by General Gilmore, it was concluded to dress the blocks up with neat cement, and, after setting of the same, to bring them to true surfaces by scraping with steel straight-edges. This treatment involves somewhat more labor than the plaster dressing, but the results were mostly satisfactory and the difference in expense is not material. Generally speaking the blocks adjusted themselves to the plates satisfactorily, and it is believed that there are no material errors in the tests due to lack of uniformity of distribution of pressure over the entire area compressed. One or two cases, however, are noted in column (22) of Table No. 2.

In reducing the results to pressure on the square inch, the blocks have been assumed as exactly one foot area. The molds were very carefully and strongly made, as already noted, of two-inch oak, and the variations from a square foot area were so very slight that no material error arises from such assumption.

In order to determine the relative value of the Genesee shales for concrete in comparison with the harder Ridge, Portage and Nunda stones, it was considered desirable to make a number of tests of the actual amount of decrease of length which a number of the blocks underwent with given pressures applied. For this purpose a simple multiplying gage, the short arm of which was 4 inches in length, and the long arm 60 inches, was set in

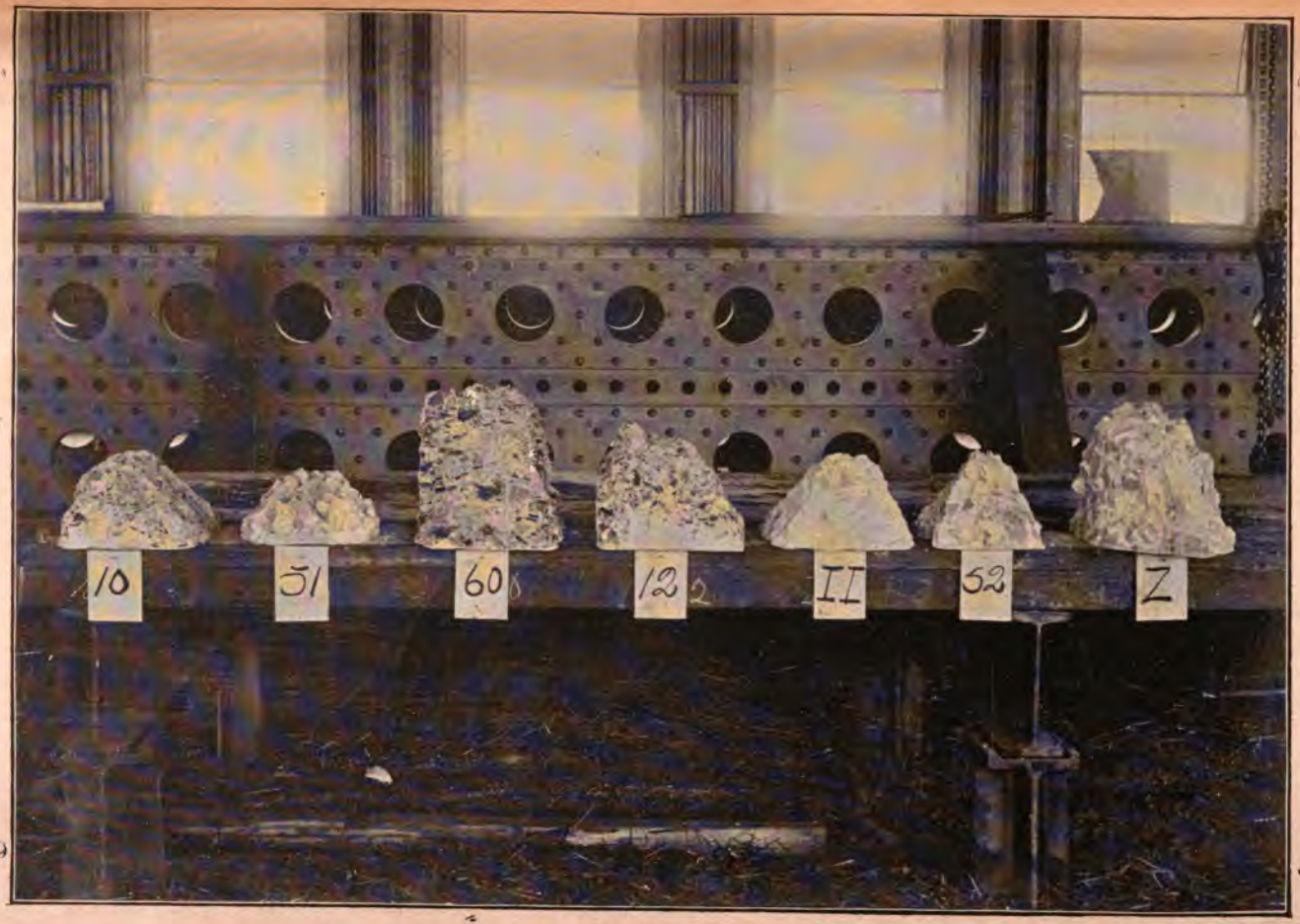


Plate I.— Illustrating Pyramidal Fracture of Concrete Blocks.

between the iron plates transmitting the pressure from the machine to the blocks. The proportionate lengths of the arms was as 1 to 15, whence the record made by the pointer at the end of the long arm was 15 times the actual decrease in length. A fine scale graduated to one one-hundredth of an inch was used to measure the distance passed over by the pointer in each case, and a division of the indicated quantity by 15 gave the actual compression. In this way the results tabulated in column (21) of Table No. 2 have been obtained.

Without going into an elaborate analysis of these compression tests it will be sufficient to state that as a whole they indicate: (1) That the compression decreases as the pressure per unit of area increases. (2) That the shale blocks compress about one and six-tenths times as much as those from the harder Ridge, Portage or Nunda stone. Table No. 3 is designed to show the relations of the two classes of stones, as regards compression, in detail.

In compiling Table No. 3, a few of the compression records, between 200 pounds on the inch and 650 pounds, have been omitted, as being so far away from the mean as to appear abnormal. For instance, it will be observed on referring to Table No. 1, that block No. 12, composed of a mortar of Wayland cement and sand 1 to 2, and shale, with mortar 43 per cent of the measured quantity of shale, gave no compression at all between the limits of 200 pounds on the square inch and 650 pounds. From 650 pounds to 1,100 the compression was 0.004 inches, and from 1,100 pounds to 1,569, the point of first crack, the compression was also 0.004 inches. And while it may possibly be true that this block was actually incompressionable between the limits of 200 pounds on the square inch and 650, still, viewing the tests as a whole and especially taking into account that the mean of the fourteen shale blocks, included in Table No. 3, is 0.01346 inches between the limits considered, it must be considered doubtful if such an indicated record is entirely correct. It has, therefore, appeared allowable to reject this determination from the mean.

Again, block 55, composed of Wayland Portland cement and sand, 1 to 4, and Nunda stone, with mortar, 42 per cent of the aggregate, gave a compression, between 200 pounds on the inch and 650, of 0.03533. Also block 69, composed of Saylor's Portland cement and sand, 1 to 3, and Ridge stone, with mortar, 0.42 per cent of the stone, gave 0.02677. Rejecting these two apparently abnormally large results, and we have 0.00825 inches as the mean of 23 blocks, between the limits of 200 pounds on the inch and 650. In view, therefore, of the considerable departure of blocks 55 and 69 from the mean, it appears proper to reject them, not, however, because it is entirely clear that the record is incorrect, but because the results of the tests, as a whole, the same as in the previous case of shale block No. 12, tend to cast some doubt upon them. In this connection it is proper to say that these apparently abnormal records were noticed when obtained, and, in each case, a close examination of the compression recording apparatus failed to show any defects therein which would account for the irregular results.

Taking the means, then, as given in Table No. 3, at 0.01346 inch for 14 shale blocks, and at 0.00825 inch for 23 hard stone blocks, and it results that as a mean the compression of the shale concretes between the limits taken in Table No. 3 was 1.63 times as great as for the blocks made from the harder Ridge, Portage and Nunda stone.

Table No. 4 has been prepared in order to show the relative breaking stresses of the shale blocks in comparison with blocks of similar mortar composition made from the harder stones. These comparisons are confined to the two Portland cements for the reason already indicated, that the results of the tests are to show the entire inadaptability of the natural cements to this work.

In regard to the comparative values of the Genesee shale and the hard stones, we learn from Table No. 4:

(1.) That Wayland cement in 1 to 1 mortar, in the proportion of 33 per cent of the broken stone, gives a concrete, with the hard Ridge, Portage and Nunda stones, from 2.2 to 2.4 times as strong as the same composition of mortar with the Genesee shale.

TABLE No. 3.

Comparison of compression tests of concrete made from Genesee shale and hard stone.

SERIAL NUMBER.	Cement.		Sand — part by measure in mortar.	Broken Stone.		Amount of compression — pressure from 200 pounds on the square inch to 650 pounds, in inches.	Remarks.
	Brand.	Parts by measure in mortar.		Quality.	Ratio of broken stone to mortar.		
2	Wayland	1	3	Shale	0.67	0.01200	
3	do	1	3	do	0.83	0.01133	
4	do	1	2	do	0.53	0.00733	
5	do	1	4	do	0.80	0.02000	
6	do	1	4	do	0.53	0.01267	
9	do	1	5	do	0.53	0.02800	
10	do	1	5	do	0.43	0.01267	Sum of 14 shale blocks = 0.18843, and $\frac{0.18843}{14} = 0.01346$
39	do	1	1	do	0.33	0.00800	
60	Saylor's	1	1	do	0.42	0.01733	
TT	do	1	1	do	0.42	0.00153	
63	do	1	2	do	0.42	0.00400	
WW	do	1	2	do	0.42	0.01133	
67	do	1	3	do	0.42	0.02547	
68	do	1	3	do	0.42	0.01677	
Mean						0.01346	
40	Wayland	1	1	Ridge	0.33	0.00800	
41	do	1	1	Portage	0.33	0.01000	
42	do	1	1	Nunda	0.33	0.00833	
47	do	1	6	Portage	0.42	0.00667	
48	do	1	2	do	0.42	0.01400	
49	do	1	2	Nunda	0.42	0.01333	
51	do	1	3	Portage	0.42	0.01067	
52	do	1	3	Nunda	0.42	0.00833	
53	do	1	4	Ridge	0.42	0.00667	
61	Saylor's	1	1	do	0.42	0.01067	
UU	do	1	1	do	0.42	0.00733	
62	do	1	1	Portage	0.42	0.00633	
VV	do	1	1	do	0.42	0.00533	Sum of 23 hard-stone blocks = 0.18966, and $\frac{0.18966}{23} = 0.00825$
64	do	1	2	Ridge	0.42	0.00900	
XX	do	1	2	do	0.42	0.00867	
YY	do	1	2	Portage	0.42	0.00333	
66	do	1	2	Nunda	0.42	0.01067	
ZZ	do	1	2	do	0.42	0.00167	
CCC	do	1	3	Ridge	0.42	0.01200	
70	do	1	3	Portage	0.42	0.00933	$\frac{0.01346}{0.00825} = 1.63$
DDD	do	1	3	do	0.42	0.00400	
EEE	do	1	3	Nunda	0.42	0.01533	
73	do	1	4	Portage	0.42	0.00500	
Mean						0.00867	

(2.) That Wayland cement, in 1 to 2 mortar, of the proportions indicated in the table, gives, with the hard stones, from 1.1 to 1.5 the strength of the same mixture with the shale.

(3.) That Wayland cement, in 1 to 3 mortars, of the proportion indicated in the table, gives, with the hard stones, from 1.4 to 1.6 the strength of the same mixture with the shale.

(4.) That Wayland cement, in 1 to 4 mortars, of the proportions indicated in the table, gives, with the hard stones, from 1.5 to 1.7 the strength of the same mixture with the shale.

(5.) That Wayland cement, in 1 to 6 mortar, of the proportions indicated in the table, gives, with the hard stones, from 2.0 to 2.3 times the strength of the same mixture with the shale.

(6.) That Saylor's Portland, in 1 to 1 mortars, in the proportion of 42 per cent of the broken stones, gives a concrete, with the hard stones, from 2.0 to 2.1 times as strong as the same composition of mortar in the same proportion with the Genesee shale.

(7.) That Saylor's Portland, in 1 to 2 mortar, in the proportion of 42 per cent of the broken stone, gives a concrete, with the hard stone, from 1.3 to 1.6 times as strong as the same composition of mortar in the same proportion with the Genesee shale.

(8.) That Saylor's Portland, in 1 to 3 mortar, in the proportion of 42 per cent of the broken stone, gives a concrete, with the hard stones, from 1.5 to 1.9 times as strong as the same composition of mortar in the same proportion with the Genesee shale.

(9.) That Saylor's Portland, in 1 to 4 mortar, gives, with the hard stone, a concrete from 1.7 to 1.8 times as strong as with the shale.

(10.) That Saylor's Portland, in 1 to 5 mortar, gives, with the hard stone, a concrete from 2.0 to 2.2 times as strong as with the shale.

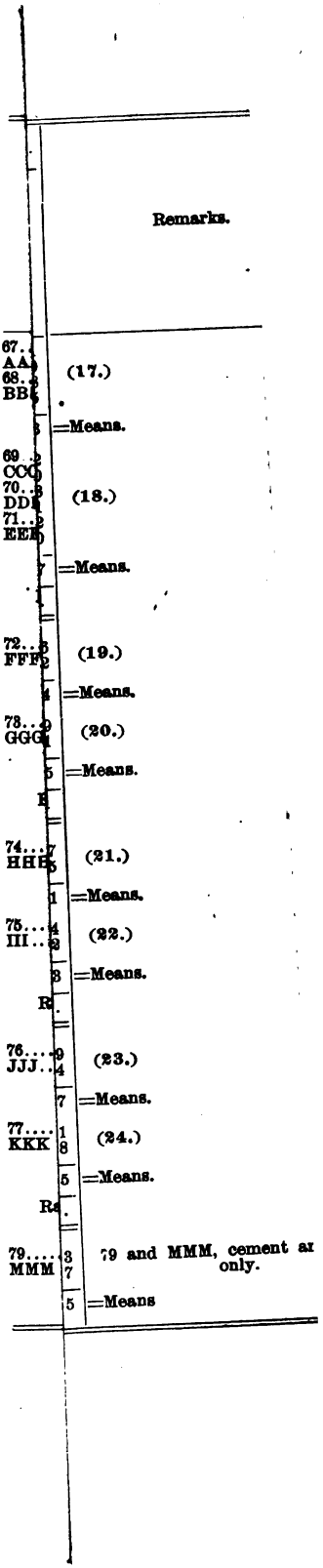
(11.) That Saylor's Portland, in 1 to 6 mortar, gives, with the hard stone, a concrete from 1.2 to 1.7 times as strong as with the shale.

Taking the indications of these tests and the nearest approach to an equality of strength of the shale and hard stone, concrete is found with a 1 to 2 mortar for both of the Portland cements

tested. For the Wayland, the relation is from 1.1 at the first crack, to 1.5 at the final stress; while for the Saylor's, 1 to 2, the relative strength is from 1.3 at the first crack to 1.6 at the final stress.

A considerable amount of concrete will be used in the proposed Genesee river storage dam for filling deep cross trenches in order to intercept the passage of water through the strata underneath the dam. The more nearly impermeable a concrete used for this purpose can be made the better it is adapted to the purpose for which it is used. Such a concrete should necessarily be as dense as the materials of which it is composed will admit, a condition only attained when the voids in the sand are entirely filled with the cement paste and the voids in the stone entirely filled with the mortar. An excess either of cement over the amount required to fill the voids in the sand, or of mortar over the amount required to fill the voids in the stone, makes the concrete more permeable and hence less valuable for stopping water. With the voids in the sand about 31 or 32 per cent of the total volume, a 3 to 1 mortar nearly fulfills the one necessary condition of impermeability. Probably, however, the point of greatest impermeability, so far as the mortar is concerned, lies somewhere between the proportion of 2 to 1 and 3 to 1, and it is satisfactory to observe, in the case of the Genesee shales, that the point of greatest strength of concrete is in close proximity to the point of greatest impermeability. This is of considerable practical importance, as the shale concretes can probably be used for making water-tight work in places where neither tensile nor compressive strength is required.

Taking the tests as a whole, and it appears that the final stresses of the blocks set in water are higher than those left in air, although there are exceptions to this, as will be seen on inspecting in detail either Table No. 2 or Table No. 5, which specially relates to this division of the subject discussed. Table No. 5 includes all the Wayland cement blocks from 39 to 55 inclusive, together with No. 78, and its air duplicate, LLL; it also includes all the Saylor's Portland blocks from 60 to 77 inclusive, together with No. 79, and its air duplicate, MMM; Nos. 78, LLL, 79 and MMM, it will be noticed, are composed of sand and cement



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Stress. Greatest pressure, net tons on the foot.	Remarks.
50.0 34.7 48.4 46.8	(9.)
45.0	=Means.
71.0 49.2 156.5 135.5	(10.)
103.1	=Means.
.....	
322.7 308.2	(11.)
315.5	=Means.
177.5 169.4	(12.) { 43, CC, 78 cement and
173.5	=Means.
98.4 108.1	(13.)
103.3	=Means.
248.5 217.8 225.1 166.2	(14.)
214.4	=Means.
.....	
98.4 75.8	(15.)
87.1	=Means.
158.1 121.0 151.7 135.5 146.8 108.1	(16.)
136.9	=Means.
.....	

only, without any broken stone, as are also blocks 43 and CC of the Wayland series.

The mean of the final stresses of all the Wayland cement blocks set in water is found to be 2,063 pounds on the square inch; while for the duplicate air blocks it is 1,912 pounds on the inch, a mean excess of strength in favor of the blocks set in water over those left in air of 8 per cent. For the Saylor's Portland blocks the corresponding figures are: Water blocks, 1,578 pounds on the square inch; and for the air blocks 1,280 pounds. In this case the mean excess of strength of the water blocks over those left in air is 22 per cent.

In considering these results, two other points should be remembered:

(1.) That with one or two exceptions, which are confined to the natural cement blocks, the water and air blocks are exact duplicates (this is true of all the Portland cement blocks) both having been molded at the same time and from the same batch of concrete.

(2.) That the air blocks are with one or two exceptions lighter than the water blocks, as will be observed by referring to column (16) of Table No. 2.

In any work like the Genesee river storage dam, where, of necessity, concrete masonry must remain for a long time in air before either coming in contact with or being submerged in water, obviously that cement is preferable which comes nearest to developing its possible maximum strength under the ordinary conditions of actual practice, whatever they may be, rather than under the special conditions of a carefully regulated experimental test. From this point of view the tests show the superiority of the Wayland cement over the Saylor's Portland in a very marked degree, and while it is not considered that this point is absolutely established as between these two cements, still it is important enough to justify mentioning as decidedly worth attention in selecting a cement for any important work. On this point it may be also further remarked that the basement in which the air blocks were stored, until taken therefrom for shipment on January 26, 1894, was an ideal place for such storage. The tanks for immersion of the water blocks were also located therein and their

large water surfaces, together with the leakage, which kept the cement floor of the room nearly covered with water during the whole period, furnished such a moist air as the conditions for development of the greatest possible strength in air demand. This basement was maintained at a uniform temperature of 60 degrees F. during the whole period of storage therein. The car in which the blocks were shipped was also fitted with a stove and the temperature maintained as nearly as possible at 60 degrees F., until the blocks were finally taken to the testing room for breaking. The length of time the water blocks were out of the water and in air of a temperature of about 60 degrees F. in the car, may be seen in detail by reference to columns (12) and (14) of Table No. 2.

The means of 2063 and 1912 for water and air for the Wayland concretes, and of 1578 and 1280 for the Saylor's Portland, as given in Table No. 5, also indicate a superiority of the Wayland over the Saylor's. On this point it may be observed that while the Wayland blocks, as whole, do show somewhat higher than the Saylor's, still a final conclusion could only be drawn properly from a comparison of the same number of blocks of the same composition. In this view Table No. 5 makes a more favorable showing than it otherwise would for the Wayland blocks. The relative ages may also be taken into account in any final decision. It may be pointed out, however, that the indication of the concrete tests is, in this particular, in accord with that of the tensile tests as shown in Table No. 1.

The fractures developed by these blocks in breaking were for the strong ones, in accordance with the observations of other experimenters, pyramidal in form. As no special practical result is deducible from the observations made on this point, the subject is not pursued any further than to call attention to plates I and II, illustrating a few of the typical forms which were photographed at the time of the tests. On plate II, however, for block 39, read 79.

In regard to the pressures producing first crack and final stress, as given in Table No. 2, it may be stated that the friction of the testing machine has been allowed for in the reductions; the

TABLE No. 5.

Relative strength of concrete blocks set in water and in air.

In Water.		In Air.		Cement.			Broken Stone.		Remarks.
Serial number.	Final stress.	Serial letter.	Final stress.	Brand.	Parts by measure in mortar.	Sand, parts by measure in mortar.	Quality.	Ratio.	
39....	1,569	Y.....	1,300	Wayland .	1	1	Shale ...	0.33	
40....	3,709	Z.....	3,339	do .	1	1	Ridge ..	0.33	
41....	2,958	AA....	3,339	do .	1	1	Portage.	0.33	
42....	2,689	BB....	2,891	do .	1	1	Nunda..	0.33	
43....	4,482	CC	4,280	do .	1	1	.....	....	Sand and cement only. No broken stone.
44....	695	DD ...	482	do .	1	6	Shale ...	0.33	
45....	986	EE....	683	do .	1	6	Portage.	0.33	
46....	672	FF ...	650	do .	1	6	Shale ...	0.42	See note in col. 22 of Table No. 2 relating to 46 and 47.
47....	2,174	GG ...	1,882	do .	1	6	Portage	0.42	
48....	2,582	HH...	2,555	do .	1	2	Portage.	0.42	
49....	2,398	II.....	2,331	do .	1	2	Nunda..	0.42	
50....	1,602	JJ....	1,210	do .	1	3	Ridge ..	0.42	
51....	1,960	KK ..	1,658	do .	1	3	Portage.	0.42	2083
52....	2,308	LL ...	1,681	do .	1	3	Nunda..	0.42	——— = 1.09
53....	1,636	MM...	1,367	do .	1	4	Ridge ..	0.42	1912
54....	1,524	NN ...	1,322	do .	1	4	Portage.	0.42	
55....	1,233	OO ...	1,098	do .	1	4	Nunda..	0.42	
78....	2,465	LLL..	2,353	do .	1	2	.....	....	Sand and cement only. No broken stone.
	2,083	.....	1,912	.....	....	....	.....	....	= Means.
60....	1,367	TT....	1,501	Saylors...	1	1	Shale ..	0.42	
61....	3,451	UU ...	3,025	do ...	1	1	Ridge ..	0.42	
62....	3,126	VV....	2,308	do ...	1	1	Portage.	0.42	
63....	1,367	WW ..	1,053	do ...	1	2	Shale ...	0.42	
64....	2,127	XX....	1,681	do ...	1	2	Ridge ..	0.42	
65....	2,107	YY ...	1,882	do ...	1	2	Portage	0.42	
66....	2,039	ZZ....	1,501	do ...	1	2	Nunda..	0.42	
67....	807	AAA..	762	do ...	1	3	Shale ...	0.42	
68....	941	BBB ..	784	do ...	1	3	Shale ...	0.42	
69....	1,613	CCC ..	1,389	do ...	1	3	Ridge ..	0.42	
70....	1,636	DDD..	1,501	do ...	1	3	Portage.	0.42	
71....	1,725	EEE..	1,389	do ...	1	3	Nunda..	0.42	
72....	661	FFF ..	627	do ...	1	4	Shale ...	0.42	
73....	1,109	GGG..	1,154	do ...	1	4	Portage.	0.42	
74....	482	HHH..	498	do ...	1	5	Shale ...	0.42	1577
75....	1,255	III ...	919	do ...	1	5	Portage.	0.42	——— = 1.19
76....	471	JJJ...	381	do ...	1	6	Shale ...	0.42	1327
77....	807	KKK..	650	do ...	1	6	Portage	0.42	
79....	2,880	MMM..	2,219	do ...	1	2	.....	....	Sand and cement only.
	1,577	.....	1,327	.....	....	....	.....	....	= Means.

Plate III.-- Conditions of Concrete Blocks Exposed to Freezing on March 17, 1894.

quantities under these heads, therefore, represent the actual pressure, as nearly as they can be obtained from the Phoenix Company's machine.

An experiment was made in a very rough way as to whether these concretes would stand winter weather when freshly made. For this purpose the surplus quantities of each mixing were made into a series of cubes of three feet, these cubes being built up in thin layers, well rammed, from day to day, as the tests went on. Plate III is from a photograph showing the appearance of these blocks March 17, 1894. Block No. 1 was composed of Wayland cement for the first foot at the bottom, with the balance natural. No. 2 was about the same, except there was a layer of Portland cement concrete about one inch thick at the top. Block No. 3 was natural cement concrete, except a layer of Portland cement concrete six inches thick, about eight inches from the top. No. 4 was composed entirely of Portland concrete, except a layer of natural cement concrete about six inches thick and about six inches from the top.

In Block No. 3 the layer of Portland concrete appears thoroughly indurated and, so far as can be observed, entirely uninjured by frost.

In Block No. 4 the Portland cement concrete is uninjured, while the layer of natural cement near the top has crumbled the same as in the other blocks.

The boxing was allowed to stand around all these blocks until after the completion of the tests in December. They were, therefore, protected from the action of storms at the sides until after thorough setting had taken place. The air temperatures during the days when these blocks were in process of making are shown in column (3) of Table No. 2. The nights were, of course, much colder.

The test by exposure to freezing indicates one point wherein the Portland cements as a class are superior to the natural, namely, they may be laid in moderately cold weather and, provided as small an amount of water as possible is used, no ill-effect will be observed. The water is the destructive agent in freezing weather, and by reducing it to a minimum there will remain, after chemical

combination, little or none to disrupt. This practice, too, will be of advantage by adding to the strength of the mixture, as it is now well understood that an access of strength of the concrete follows a reduction of the amount of water used in mixing; for the greatest strength the smallest possible quantity of water should be used. For very great strength the quantity of water may be so far reduced as to cause the mortar to resemble damp earth. When such mortar is added to broken stone, which have first been well wetted and allowed to drain, the water may by thorough ramming be still flushed to the surface in a very thin film.

In freezing weather, therefore, the practice which prevents injury to the work will also be the proper practice for developing the greatest possible strength.

It has been a common practice to add salt to the water used in mixing in freezing weather in the proportions of about one per cent of saturation for each degree below the freezing point. The weight of evidence is that for well proportioned Portland cement this practice is entirely wrong — much better results will be obtained by the practice indicated in the foregoing.

We come, now, to the final point of the study, namely, to answer the questions proposed at the beginning as to the adaptability of the various materials tested for use in concrete in the construction of the proposed Genesee storage dam. The formal answers are as follows:

(1.) As regards the Genesee shales, they are not suitable for the main dam, or any portion of it which will be subjected to severe strains. At present there appears to be no reason why they can not be used for filling cross-trenches and for sheeting the sides and bottom, where the only object to be attained is the making of impermeable work.

Inasmuch as there is a considerable quantity of such work estimated, their use, even when restricted to this one feature, will be the source of an economy of from $40,000 to $50,000.

(2) As regards the Ridge, Portage and Nunda stones, the tests show that all of them are suitable for the aggregate of high-grade concrete. There is no good reason why the proposed storage dam can not be mostly constructed of a concrete fabricated from some

TABLE No. 6.

Results of compression tests of Pennsylvania, Ridge, Portage and Nunda stone.

QUALITY OF STONE.	Serial number.	Dimensions of block.	Weight of block in pounds.	Weight per cubic foot in pounds.	First Crack.		Final Stress.		Explanations.
					Pounds on the square inch.	Net tons on the block.	Pounds on the square inch.	Net tons on the block.	
Pennsylvania sandstone.....	1	6"×12"×12"	71.5	143.0	4,280	308.2	6,678	480.8	This block was dressed on all its faces. Pressure applied to the 144-inch faces at right angles to beds. Broke with loud report.
Pennsylvania sandstone.....	2	6"×12"×12"	71.75	143.5	.....	....	4,258	153.3	Dressed on all faces. Pressure applied to 72-inch faces parallel to beds. Pieces cracked off before pressure reached 95 tons.
Pennsylvania sandstone.....	3	6"×12"×12"	81.0	162.0	2,936	211.4	9,166	659.9	Dressed on 144-inch faces only. Rock face on balance of sides. Pressure applied on 144-inch face at right angles to beds.
Pennsylvania sandstone.....	4	6"×12"×12"	81.25	162.5	3,115	224.3	10,129	729.3	Same as (3).
Pennsylvania sandstone.....	5	12"×12"×12"	142.0	142.0	2,219	159.7	4,437	319.5	Dressed on all faces. Pressure applied at right angles to beds.
Ridge.....	6	9"×9"×9"	70.25	166.5	3,147	127.5	8,286	335.6	Dressed on all faces. Pressure applied at right angles to beds.
Portage	7	12"×12"×12"	155.0	155.0	4,460	321.1	7,709	555.0	Dressed on all faces. Pressure applied at right angles to beds
Nunda.....	8	8"×12"×12"	106.0	159.0	4,347	313.0	5,916	426.0	Dressed on all faces. Pressure applied on 144-inch face at right angles to beds.

one of these stones. In connection with rubble masonry, their use for such purpose will be the source of an economy of about $300,000.

(3.) As regards the natural cements the tests show that none of them are suitable for a work of this character. Their general unreliability would of itself condemn them for use in a dam 130 feet in height. On this point reference may be made to Table No. 2, where it will be seen that out of a total number of natural cement test blocks of 78, nearly one-third of the whole, or 25, failed to give any result whatever.

Block F, as shown in column (22) of Table No. 2, was accidentally destroyed in the testing machine, and is not included in the number as failing to give any record. This statement is intended to include only those which were found to be cracked or which crumbled or were easily broken with light blows of the hammer when taken from the water. Moreover, it is significant that with one exception the failures of the natural cement blocks were all among the water blocks.

If we examine the detailed record of the Portland cement blocks as given in Table No. 2, we find that out of a total number of 96 made, not one failed to give a useful result in the tests. This fact alone establishes beyond all question the great superiority of the Portland cements over the natural for any work requiring a reliable material under any and all circumstances.

In reference to these two classes of cement it may be stated that every attempt was made to get the best results in these tests which the several materials were capable of yielding, and while the proportion of failures among the natural cements appears unduly large, still an examination of the records, for two or three years back, in the State testing office, easily convinces one that these results are on the whole not specially exceptional. In making this statement it is not intended to assert that about one-third of all the tensile tests of natural cements are failures, although the tests of the same cement frequently shows such great variations from different samples as to easily lead to a positive conclusion as to their extreme unreliability. In addition the records show a considerable proportion of absolute failures.

In the present case it may be concluded, too, that the water bath was a severe test for concrete blocks made from the natural cements.

The conclusion is, then, that the two American Portland cements tested are suitable for the work in hand. Their relative values have been discussed to some extent throughout the report.

(4.) The detailed tensile tests of Table No. 1 show that the Mt. Morris sand is of a quality suitable for an important work of this magnitude.

In using concrete on a large scale, as in a work of this character, mechanical mixing will preferably be used, not only on the score of economy of labor, but on account of yielding more thorough results. As we have seen, it is important that the mixing proceed according to a definite formula, in order that such results may be secured. The necessary plant for this purpose would properly include:

(1) A series of bins for storing cement, sand and broken stone together with a tank for storing water, from all of which the proper quantities of each may be fed to the mixing machine by arrangements under the control of one person.

(2) A mixing machine which admits of (a) the dry mixing of the cement and sand in the proper proportions; (b) the thorough mixing of the cement mortar, after the addition of the water; and (c) the separate mixing of the mortar with the aggregate.

(3) Mechanical arrangements for immediately carrying the concrete in quantity from the mixing machine to its final place in the work.

The delivery of all the material to be used in construction at the top of the high bank, in the manner proposed in the preliminary report would render feasible arrangements embodying these conditions at a minimum of expense.

As a final point in the concrete study it may be stated that after reviewing the whole subject in its various phases the present writer has arrived at the conclusion, that, in order to provide for expansion and contraction in the direction of the length, the work should be built in large blocks placed far enough apart to admit of putting up and taking down the boxing; the spaces

between, to be filled later on with concrete thoroughly rammed. The large blocks may be set to break joints both vertically and horizontally, thereby avoiding horizontal joints across the whole width of the wall.

Concluding the subject of concrete study, it may be stated that the tests, as a whole, while giving many valuable results, are hardly as complete as could be desired. They lack somewhat in symmetry, due not only to the large number of points requiring investigation, but to the small amount of money as well as time at my disposal for this part of the work. In making them as they stand, it was assumed, that in case the project was carried out, and it was concluded to use concrete for the main portion of the work, an opportunity for studying any questions requiring further consideration would be afforded. It is, therefore, suggested, in case this work goes on, that additional studies in reference to the economical limit of the quantity of mortar in relation to the voids in the aggregate be made.

For such final studies it seems unnecessary to go outside of the Portland cements. So far as can be foreseen, the cost of the desirable additional studies would be small. The present tests have not cost to exceed about $1,500.

Some of the more interesting results of the tests are shown graphically in Plates IV and V, which are self-explanatory.

A few compression tests have also been made of the Pennsylvania sandstone and of blocks from Ridge, Portage and Nunda quarries. The results are given in Table No. 6. Blocks Nos. 1 to 5, of the Pennsylvania sandstone, were all broken with pressure at right angles to the beds, except No. 2, on which the pressure was applied parallel to the natural beds. Blocks 3 and 4 were dressed on the beds and left rock face at the sides, which accounts not only for the excessive weight of these two blocks, but, probably, for the large final stress. The Pennsylvania sandstone are rather coarse grained.

Block No. 6, a nine-inch cube from the Ridge quarry, actually weighed 70.25 pounds, which gives a calculated weight per cubic foot of 166.5 pounds. The final stress on this block was 8,286

pounds on the square inch. The important points of the Portage and Nunda blocks are sufficiently shown by the tabulation.

We may now take up the detail of the water pressure tests, of which a general description has been given in the preliminary report. The following condensations from the daily journal of these tests will serve to illustrate the methods and results obtained:

September 21, 1893.— Finished first drill hole, XX3, to depth of 42 feet in rock, or to 67.9 feet from top of casing (elevation of bottom of drill hole, 524.7). At depth of 7 feet in the rock (elevation 561.7) the wash water disappeared — that is, stopped returning to the surface through the casing, as it should, so long as everything below is water-tight. The inference is that there are seams through which the water runs away, although it may possibly escape under the shoe at the top of the rock. The appearance of the drill rods, however, was such as to indicate the presence of strong sulphur water. After removing drill the water was found standing in casing at 12 feet below surface (stood at elevation of 580.6) and could not be made to stand permanently higher, even when forced down in quantities under pressure from pump. A common suction pump was set 25 feet down (at elevation of 567.9). On pumping at rate of 5 to 6 gallons per minute, water in hole soon fell several feet. After about 15 minutes the water pumped showed strong with sulphur, and was also deeply discolored with the drill cuttings. This apparently indicated not only the presence of seams containing sulphur water, but that the escape of the wash water, referred to in the foregoing as taking place while drilling, had been into seams from which when the action was reversed, the cuttings were easily brought back.

September 26.— In afternoon set seed bag two feet long in drill hole XX3. (Lower end of bag at elevation 564.6. Top of rock, 568.7.)

Wednesday, September 27.— Connected pump with one-inch pipe leading down through seed bag and ran at full capacity with 60 pounds of steam. Water pressure at surface of ground, as indicated by gage, 20 pounds. Water at this pressure was forced below seed bag for an hour, apparently without effect. The conclusion is that at some point below elevation 564.6 the rock is open enough to allow water to run away.

September 30.— Made further tests of drill hole XX3. Seed bag had been set to 40 feet below top of casing (at elevation of 552.6, or 16.1 feet below top of rock). Pressure as indicated by gage at surface of ground, 32 feet from top of casing, 22 pounds. Total length of one-inch pipe beyond pressure gage, 76 feet.

Under these conditions hole took full capacity of pump with 60 pounds steam pressure for one-half hour without showing any increased pressure.

October 6.— Tested drill hole ZZ3. Seed bag, 35 feet down (at elevation of 554.0). Pressure gage at surface of ground showed 5 pounds pressure. Seed bag leaked and test was in consequence without definite results.

October 7.— Again tested ZZ3, with same results as on previous day. Thereupon pulled out old seed bag and set new one.

October 9.— Again tested ZZ3. Found seed bag tight. With 60 pounds steam pressure and full capacity of pump, gage showed 20 pounds. When pump was first started water rose in casing to surface of ground and overflowed, but quickly dropped back several feet. There seems to be no doubt but that seed bag was tight this time and that, therefore, the water must have come up through crevices in the rock around the seed bag.

The preceding tests having shown that the seed bag was too cumbersome for this work, a rubber packer was procured and the balance of the tests made with its assistance. This was so arranged, with stand pipe leading to bottom of hole, as to instantly pack tight on the application of pressure at the surface of the ground. By its use definite results were obtained, as described in the preliminary report, which would have been impossible of attainment with the seed bag.

October 17.— Tested drill hole YY4, using rubber packer, which was set 50 feet from top of casing (at elevation of 539.5). On first starting pump gage showed 60 to 70 pounds; pressure gradually rose to 110 pounds. At this latter pressure the hole took all the water the pump could deliver.

Before starting pump, put in hole six ounces of aniline red and eleven ounces of permanganate of potash. Stationed men on river in boats but could find no trace of this color.

After pumping for an hour with packer at elevation 539.5, disconnected and found that water ran slowly from top of pipe about 5 feet above surface of ground (elevation 594.0), thereby showing that a small head had been gained at the sides.

Packer was then raised to an elevation of 559.5 (top of rock at 565.2), a pound of aniline red added and pump again connected. With pump at full capacity, pressure gage now showed only 20 pounds pressure, and no more could be gained however rapidly pump was run. The clear inference is that between elevations 539.5 and 559.5 there are seams or fissures which allow water to flow out of drill hole when under about 20 pounds pressure.

No trace of the coloring matter could be found along the river in either case.

Judging from the indications of the water pressure tests, as well as from the condition of the core, the first 10 to 15 feet of rock at Dam Site No. 2, especially on the east side, is somewhat seamy and broken up (see the cores in verification of this view).

October 19.— Tested drill holes ZZ1. Packer was first set 19 feet above bottom (elevation of 532.5), stalled pump at 100 pounds pressure. Raised packer gradually, packing it at every few feet by setting pressure above against pressure of water from below. In this way the hole was tested for its entire length and found to stand, as stated, 100 pounds at the bottom, and from 40 to 50 pounds in the upper part. Best results in the upper section of any yet tested.

October 31.— Tested horizontal hole at foot of Hogback. Set packer 87 feet in., or eight feet from end. On starting pump, gage showed 100 pounds and gradually rose to 140 pounds. Released packer and stopped every 10 feet until 54.5 feet from bottom was reached. At this point gage dropped to 100 pounds, but in one-half hour again advanced to 140 pounds, and finally in 15 minutes more to 150 pounds, where it remained for one-half hour and then dropped to 60 pounds, steam remaining the same. In an hour and forty-five minutes pressure showed 40 pounds. At this point water dripped from the side of the Hogback for some distance to the south.

November 2.— Repeated this test with packer 55 feet from bottom of hole, and with four quarts of wheat bran below packer. Pumped with 100 pounds pressure for two hours without effect.

November 2.— Tested hole 23 + 42 W. 250, at Hogback location. Hole 84 feet deep, 14 feet to rock (elevation of bottom 503.4; top of rock 571.4). Set packer 8.5 feet from bottom. Gage showed 165 pounds. Raised packer to elevation 524.4 and pressure dropped to 80 pounds. Disconnected and put in four quarts of bran, whereupon pressure rose to 170 pounds. Raised packer to elevation 534.0 and still maintained same pressure. At elevation 536.0 pressure dropped to 100 pounds and remained at that point even after the addition of 5 quarts of bran. Pressure finally fell to 60 pounds and remained there for three and one-half hours. While pumping this hole water ran from casing at 23 + 42 W. 350.

Also put two pounds of permanganate of potash in hole, no trace of which could be discovered along the river.

November 4.— Tested 23 + 42 W. 350 at Hogback location. Set packer 10 feet from bottom (elevation of 518.0), got 40 pounds pressure on gage. Added

4 quarts of bran without effect. on the pressure. In two and one-half hours the pressure gradually rose to 65 pounds. Coloring matter was added, and showed in water flowing from casing at hole 23 +42 W. 250. On stopping pump it was found that water pumped into hole had acquired a back pressure of 20 pounds. On disconnecting, water ran from pipe for one hour and forty-nine minutes. This outflowing water showed coloring matter for fifteen minutes.

This test indicates not only a connection between holes 23 + 42 W. 250, and this one under the river bed, and independent of it; but also shows backing up, probably in vertical seams, at the sides of the gorge. A number of other tests at this site gave the same result.

November 24.—Tested B. 40 + 70 W. 750 at Dam Site No. 1. Set packer 6 feet from bottom. Pressure rose to 180 pounds, pump stalled. Raised packer 10 feet, or to 16 feet from bottom, when pressure at first rose to 170 pounds, but in a few minutes fell to 120 pounds, where it remained for ten minutes, and finally fell to 100 pounds. Uncoupled and added bran, when pressure rose from 100 to 120 pounds. Raised packer to 26 feet from bottom and gage showed 40 pounds. Again added bran and gage rose to 50 pounds. With packer at this elevation gas issued from casing at hole B. 40 + 70 W. 850. Upon raising packer 2 feet more, larger quantities of gas flowed from that hole. The packer was raised and lowered several times, with like results, showing a connection between the two holes at about elevation of 548.0.

The foregoing are the details of a number of the pressure tests. The general deductions from these various tests have been so fully given in the preliminary report as to make further discussion of them unnecessary here.

In regard to the estimates of cost of which the footings are submitted in the preliminary report, it may be stated that, on revision, it appears the footings for all the rubble masonry dams estimated for Sites 1 and 2 could be reduced somewhat, but the estimate for an earth dam at the Hogback would, perhaps, need to be increased. The results of the concrete tests, as already exhibited, are, however, so satisfactory, and lead, with other modifications of plan, to such material reductions of cost as to render further consideration of either an earth dam at the Hogback location, or of rubble masonry dams at Sites 1 and 2 unnecessary. The proposed use of concrete construction, therefore,

narrows the choice to sites 1 and 2, where the question of cost may properly be made, to some extent, subordinate to that of absolute safety and durability.

The following are the detailed estimates for a series of dams at these two sites to be built either wholly, or nearly wholly, of concrete, or with rubble masonry faces and concrete cores as the case may be. As will be noted, on inspection, these estimates provide for clearing the timber from the whole reservoir and for the construction of the necessary transportation line.

Estimated Cost of Concrete Dam at Site No. 1, 130 *Feet in Height.*

Two thousand six hundred and twenty acres of land, at $30 per acre	$78,600
Nine hundred acres of clearing, at $20	18,000
Three acres of clearing and grubbing, at $100	300
Changing bridge and highway at St. Helena	7,500
Bailing and draining whole work	75,000
Grouting in foundation	30,000
One hundred thousand cubic yards foundation excavation (earth), at 30 cents	30,000
Sixty-six thousand cubic yards foundation, excavation (rock), at $1	66,000
Fifteen thousand cubic yards, granite masonry, at $20	300,000
Eighty thousand cubic yards concrete, at $5.50	440,000
One hundred and ninety thousand cubic yards concrete, at $4	760,000
Thirty thousand cubic yards concrete, at $3	90,000
Seven thousand cubic yards rubble masonry, at $7	49,000
Six hundred and fifty cubic yards coping, at $40	26,000
One thousand three hundred and fifty cubic yards coping, at $20	27,000
One thousand cubic yards clay puddle, at $1	10,000
Three thousand five hundred cubic yards riprap (large stone), at $4	14,000
Gate-house building	20,000

Gates and operating machinery	$40,000
Transportation line	50,000
Add for contingencies and superintendence	318,600
Amount	$2,450,000

With sandstone faces throughout, except for the spillway, where granite is provided, and concrete core, the estimated cost of this dam would be $2,590,000.

Estimated Cost of Concrete at Dam Site No. 1, *Built to Height of* 58 *Feet, but Susceptible of Extension to* 130 *Feet.*

One thousand six hundred and twenty acres of land, at $30	$48,600
Five hundred acres of clearing, at $20	10,000
Three acres of clearing and grubbing, at $100	300
Bailing and draining whole work	75,000
Grouting in foundation	30,000
Ninety-five thousand cubic yards foundation excavation (earth), at 30 cents	28,500
Fifty-nine thousand cubic yards foundation excavation (rock), at $1	59,000
Nine thousand cubic yards granite masonry, at $20	180,000
Forty-eight thousand cubic yards concrete, at $5.50	264,000
One hundred and forty thousand cubic yards concrete, at $4	560,000
Twenty-five thousand cubic yards concrete, at $3	75,000
Seven hundred cubic yards coping, at $20	14,000
Ten thousand cubic yards clay puddle, at $1	10,000
Three thousand five hundred cubic yards riprap, at $4	14,000
Temporary gate-house building	8,000
Gates and operating machinery	35,000
Three hundred thousand feet B. M. of oak timber in work, at $35	10,500
Transportation line	50,000
Add for contingencies and superintendence	228,100
Amount	$1,700,000

With sandstone faces throughout, except for the spillway, where granite is provided, and concrete core, the estimated cost of this dam would be $1,780,000.

Estimated Cost of Completing a Concrete Dam at Site No. 1, *Built Originally to Height of* 58 *Feet, but Susceptible of Extension to* 130 *Feet; Cost of Completing Between* 58 *Feet and* 130 *Feet.*

One thousand acres of land, at $30	$30,000
Four hundred acres of clearing, at $20	8,000
Changing bridge and highway at St. Helena	7,500
Five thousand cubic yards foundation excavation (earth), at 30 cents	1,500
Seven thousand cubic yards foundation excavation (rock), at $1	7,000
Six thousand cubic yards granite masonry, at $20	120,000
Thirty-two thousand cubic yards concrete, at $5.50	176,000
Fifty thousand cubic yards concrete, at $4	200,000
Five thousand cubic yards concrete, at $3	15,000
Seven thousand cubic yards rubble masonry, at $7	49,000
Six hundred and fifty cubic yards coping, at $40	26,000
Six hundred and fifty cubic yards coping, at $20	13,000
Permanent gate-house building	20,000
Additions to machinery	10,000
Contingencies and superintendence	117,000
Amount	$800,000

With sandstone faces throughout, except for the spillway, where granite is provided, and concrete core, the estimated cost of this dam would be $860,000.

Estimated Cost of Dam at Site No. 1, 100 *Feet in Height.*

Two thousand two hundred and fifty acres of land, at $30	$67,500
Seven hundred acres of clearing, at $20	14,000
Three acres of clearing and grubbing, at $100	300
Bailing and draining whole work	65,000
Grouting in foundation	25,000

Ninety-six thousand cubic yards foundation excavation (earth), at 30 cents	$28,800
Fifty thousand cubic yards foundation excavation (rock), at $1	50,000
Ten thousand cubic yards granite masonry, at $20	200,000
Forty thousand cubic yards concrete, at $5.50	220,000
One hundred and ten thousand cubic yards concrete, at $4	440,000
Twenty-five thousand cubic yards concrete, at $3	75,000
Five thousand cubic yards rubble masonry, at $7	35,000
Five hundred and ninety cubic yards coping, at $40,	23,600
Six hundred and fifty yards coping, at $20	13,000
Seven thousand cubic yards clay puddle, at $1	7,000
Three thousand cubic yards riprap, at $4	12,000
Gate-house building	15,000
Gates and operating machinery	30,000
Transportation line	50,000
Contingencies and superintendence	208,800
Amount	$1,580,000

With sandstone faces throughout, except for the spillway, where granite is provided, and concrete core, the estimated cost of this dam would be $1,650,000.

Estimated Cost of Concrete Dam at Site No. 1, 58 Feet in Height.

One thousand five hundred acres of land, at $30	$45,000
Five hundred acres of clearing, at $20	10,000
Bailing and draining whole work	55,000
Grouting in foundation	20,000
Thirty-seven thousand cubic yards foundation excavation (earth), at 30 cents	11,100
Forty-eight thousand cubic yards foundation excavation (rock), at $1	48,000
Six thousand cubic yards granite masonry, at $20	120,000
Sixteen thousand cubic yards concrete, at $5.50	80,000
Forty-seven thousand cubic yards concrete, at $4	188,000

Eighteen thousand cubic yards concrete, at $3	$54,000
Three thousand cubic yards rubble masonry, at $7..	21,000
Two hundred and fifty cubic yards coping, at $40..	10,000
Four hundred cubic yards coping, at $20	8,000
Six thousand cubic yards clay puddle, at $1	6,000
Three thousand cubic yards riprap, at $4	12,000
Gate-house building	12,000
Gates and operating machinery	25,000
Transportation line	50,000
Add for contingencies and superintendence	124,900
Amount	$900,000

With sandstone faces throughout, except for the spillway, where granite is provided, and concrete core, the estimated cost of this dam would be $930,000.

Estimated Cost of Dam at Site No. 2, 130 Feet in Height

Two thousand four hundred acres of land, at $30 per acre	$72,000
Eight hundred and fifty acres of clearing, at $20..	17,000
Ten acres of clearing and grubbing, at $100.....	1,000
Changing bridge and highway at St. Helena.....	7,500
Bailing and draining whole work.....	50,000
Grouting in foundations	40,000
Seventy-five thousand cubic yards foundation excavation (earth), at 30 cents.....	22,500
Forty-five thousand cubic yards foundation excavation (rock), at $1	45,000
One hundred and fifty thousand cubic yards earth excavation in spillway, at 18 cents.....	27,000
Four hundred and twenty-five thousand cubic yards rock excavation in spillway, at 70 cents.....	297,500
Two thousand cubic yards granite masonry, at $20..	40,000
Seventy thousand cubic yards concrete, at $5.50..	385,000
Two hundred and twenty thousand cubic yards concrete, at $4	880,000

Forty thousand cubic yards concrete, at $3.....	$120,000
Seven thousand cubic yards rubble masonry, at $7..	49,000
Two thousand cubic yards coping, at $20.....	40,000
Ten thousand cubic yards clay puddle, at $1.....	10,000
Gate-house building	20,000
Gates and operating machinery	40,000
Transportation line	45,000
Contingencies and superintendence	191,500
Amount	$2,400,000
Add water cushion wall	200,000
	$2,600,000

With sandstone faces throughout, and including water cushion wall, concrete core and independent spillway, the estimated cost of this dam would be $2,720,000.

Estimated Cost of Dam at Site No. 2, Built to Height of 58 Feet, but Susceptible of Extension to 130 Feet.

One thousand four hundred acres of land, at $30..	$42,000
Four hundred and sixty acres of clearing, at $30..	9,200
Three acres of clearing and grubbing, at $100....	300
Bailing and draining whole work	50,000
Grouting in foundation	40,000
Seventy thousand cubic yards foundation excavation (earth), at 30 cents	21,000
Thirty-eight thousand cubic yards of foundation excavating (rock), at $1.....	38,000
Two thousand cubic yards granite masonry, at $20..	40,000
Forty-five thousand cubic yards concrete, at $5.50..	247,500
One hundred and thirty-three thousand cubic yards concrete, at $4	532,000
Twenty thousand cubic yards concrete, at $3....	60,000
Ten thousand cubic yards clay puddle, at $1.....	10,000
Two hundred and forty thousand feet B. M. of oak timber in work, at $35.....	8,400

Temporary gate-house building	$8,000
Gates and operating machinery	35,000
Transportation line	45,000
Contingencies and superintendence	173,600
Amount	$1,360,000
Add water-cushion wall	200,000
	$1,560,000

With sandstone faces throughout, and with concrete core, and including water-cushion wall, the estimated cost of this dam would be $1,640,000.

Estimated Cost of Completing a Dam at Site No. 2, Built Originally to Height of 58 Feet, but Susceptible of Extension to Height of 130 Feet; Cost of Completing Between 58 Feet and 130 Feet.

One thousand acres of land, at $30	$30,000
Four hundred acres of clearing, at $20	8,000
Changing highway at St. Helena	7,500
Five thousand cubic yards foundation excavation (earth), at 30 cents	1,500
Seven thousand cubic yards foundation excavation (rock), at $1	7,000
Thirty thousand cubic yards concrete, at $5.50	165,000
Seventy-two thousand cubic yards concrete, at $4	288,000
Twenty thousand cubic yards concrete, at $3	60,000
One hundred and fifty thousand cubic yards earth excavation in spillway, at 18 cents	27,000
Four hundred and twenty-five thousand cubic yards rock excavation in spillway, at 70 cents	297,500
Permanent gate-house building	20,000
Additions to machinery	10,000
Contingencies and superintendence	178,500
Amount	$1,100,000

With sandstone faces throughout, and with concrete core and independent spillway, the estimated cost of this dam would be $1,150,000.

Estimated Cost of Concrete Dam at Site No. 2, 58 Feet in Height.

One thousand four hundred acres of land, at $30	$42,000
Four hundred and sixty acres of clearing, at $20	9,200
Bailing and draining whole work	50,000
Grouting in foundation	25,000
Forty-two thousand cubic yards foundation excavation (earth), at 30 cents	12,600
Fifty-five thousand cubic yards foundation excavation (rock), at $1	55,000
Six thousand cubic yards granite masonry, at $20 ..	120,000
Seventeen thousand cubic yards concrete, at $5.50..	93,500
Fifty-three thousand cubic yards concrete, at $4	212,000
Eighteen thousand cubic yards concrete, at $3	54,000
Two hundred and fifty cubic yards coping, at $40 ...	10,000
Four hundred and eighty cubic yards coping, at $20..	9,600
Six thousand cubic yards clay puddle, at $1	6,000
Three thousand cubic yards riprap, at $4.....	12,000
Gate-house building	12,000
Gate and operating machinery	25,000
Transportation line	45,000
Contingencies and superintendence	117,100
Amount	$910,000

With sandstone faces throughout, except for the spillway, where granite is provided, and with concrete core, the estimated cost of this dam would be $940,000.

In regard to the foregoing estimates it may be remarked that if instead of the granite facing for spillway it should be deemed safe to use a 1 to 1 beton, a still further reduction of cost of 130 foot dam at Site No. 1 of about $200,000 could be made, with proportionate reductions in cost of all the other dams, for which, in the estimates now submitted, granite-faced spillways have been provided.

An estimate for a 100-foot dam at Site No. 1 has also been prepared as given in detail on page 80. Comparison with the other estimates indicates that the cost of a dam of this height

at Site No. 2 would not differ greatly from the cost at Site No. 1. In either case it will be necessary in a dam of this height, to provide for passing the flood flows over the front, hence the general form of section, applicable to Site No. 1 and given in the preliminary report would be used.

In regard to the stepping of the spillways, it may be stated that further study indicates that for the large flood flows to be provided for in the case under consideration, it is doubtful if the stepping, as shown on the general sections of dams, applicable to Site No. 1, accompanying the preliminary report would be of any particular advantage, and it is consequently deemed better, all things considered, to use smaller blocks of granite than were estimated in the preliminary estimates, and set them with the beds at right angles to the curve of the front of the dam, at the same time dispensing with an expensive system of doweling, which was included in the original estimates. These several modifications of plan are considered sufficient to justify a reduction of the estimated price of the granite masonry from $30 per cubic yard to $20, a change which of itself reduces the estimated cost of this portion of the work about $175,000.

Again it may be pointed out that the success of a work of this magnitude will depend upon the care and intelligence of the superintendence, and all the suggestions and conclusions of this report are presented on the supposition that every part of the work receives such superintendence.

Moreover, it should be understood that while the estimates herewith submitted are believed to be amply sufficient to fully complete the work in first-class manner throughout, such result can only be attained in practice by the making of such administrative arrangements as will permit of the work going on continually from year to year with the same contractor and under the same general arrangements from beginning to end. It would appear desirable therefore to so draw the enabling act as to permit of the execution of a contract for the whole work at the original letting, rather than to rely on new contracts after each annual appropriation, as would be necessary under existing laws. So expensive a plant is required that a change of contractor from year to

year, or even the possibility of such a change, would tend to greatly increase the final cost of the work. It would, of course, be possible for the State itself to furnish the plant complete at the beginning, but this policy would be undesirable, as leading to vexatious questions in regard to maintenance and renewals not only on general principles but because of the possibility of partial or complete destruction of any part of the plant located in the river bed by floods. It will be better in every way to so arrange the contract as to permit of full assumption of all the responsibility for the necessary working plant by the contractor. As an alternative proposition, which would obviate this particular class of difficulties, the work could be constructed by day's work by the State, but this again would inevitably lead to an excess of cost over what can be attained by a well-devised contract, properly supervised.

The completion of a dam 130 feet high would require, as stated in the preliminary report, about five years time.

The time at my disposal has not permitted of any further consideration of transportation questions than that given in the preliminary report.

We come now to a discussion of the rainfall of the Genesee valley in its relation to the ordinary and flood flow of the stream. As the basis of any such discussion, Table No. 7, wherein is embodied the monthly means derived from all the records of rainfall observations that can be obtained, has been compiled. An inspection of this table will show that the rainfall year has been taken as beginning in December. The reasons for this will appear from the following general discussion, for which I am mostly indebted for the suggestion to an exceedingly valuable paper by Mr. C. C. Vermule, on Water Supply and Water Power, which appeared as a serial in the annual reports of the New Jersey Geological Survey for the years 1890, 1891, 1892.

As the basis of a storage project, we need to know (1) the minimum flow of a stream and the mean period of its continuance from year to year; (2) the maximum rate of flow during floods; and (3) the proportion of the rainfall which runs off in a year of minimum flow, together with the distribution of the same. Provided

we have sufficiently extended rainfall records, in conjunction with synchronous gagings of the flow of a stream, all these elements are at once easily known. Or gagings of a stream alone will give (1) and (2). If, however, the rainfall records are sufficiently extended we may arrive at an approximate knowledge of the elements of the problem by means of a comparative study of the rainfall and the results of gagings on other streams where the conditions are known.

The total rainfall upon a given drainage area is disposed of in the following ways:

(1.) A portion is evaporated directly by the sun.

(2.) Another portion is taken up by plant growth and largely transpired as vapor.

(3.) A third portion, large in winter and spring, but very small in summer, flows off the surface directly into the stream, forming ordinary surface or flood flows.

(4.) A final portion sinks into the ground, to replenish the great reservoir of ground water, from which plants are fed and stream flows maintained during periods when the rainfall is less, as it frequently is for months, than the combined demands of evaporation, plant growth and stream flow.

In regard to direct evaporation, so far as water surfaces are concerned, we have tolerably complete information, as derived from the elaborate experiments of Mr. Fitzgerald, at Boston, and many others. In the case of the upper Genesee river, with only one small lake (Silver lake) the proportion of water surface to total surface is so small that evaporation from water surfaces may be neglected. The data for estimating evaporation from the ground and the amount of water taken by various crops are much less extensive than that for water surfaces. The following, from Risler, gives the best available information as to daily consumption of water by various crops and two classes of forest trees:

	Inches.
Meadow grass requires from	0.134 to 0.267
Oats require from	0.140 to 0.193.
Indian corn requires from.....	0.110 to 0.157
Clover requires from	0.140 to .....
Vineyards require from	0.035 to 0.031

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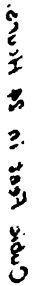


Meadow
Oats
Indian
Clover
Vineya

	Inches.
Wheat requires from	0.106 to 0.110
Rye requires from	0.091 to .....
Oak trees require from	0.038 to 0.035
Potatoes require from	0.038 to 0.055
Fir trees require from	0.020 to 0.043

Applying these figures and we learn that our ordinary farm crops may take up from twelve to fifteen inches of water over the whole area cropped during the growing period. Forests, in the same way, may take up, in their growing period, from April to August, inclusive, from four to five inches, while cleared areas which are uncultivated, probably take up from seven to eight inches.

Forests then not only use less water than cultivated areas but they further tend to increase the summer flow of streams by holding back the water in accumulations of leaves and mosses until it can be gradually absorbed into the ground. Hence the importance of a large forest area as a conservator of stream flow on any given water shed. A drainage area entirely covered with forest may yield nearly double the flow during the growing season that will be obtained from an area either entirely or nearly entirely in cultivation. Independent of the drainage of swamps and low lands generally, we derive from these figures additional reasons why the summer flow of our streams, now, is smaller than formerly.

On cultivated areas the quality of the leading crop will further materially influence the run-off. Thus, an Illinois prarie, where Indian corn may occupy 50 to 60 per cent of the total area, will give a very different summer flow from one similarly situated, where crops demanding less moisture than Indian corn are raised.

As regards the present timber area of the upper Genesee watershed, it has probably been reduced somewhat in the fourteen years since 1880. The figures of the 1890 census are, however, not yet available, and those of 1880, are, accordingly, used as the best at hand. Because of the probable reduction of forest area since 1880, we may properly increase somewhat our estimate of

the present demands for vegetation, as has been done in the final figures following:

Again, since grains, heavy grasses and Indian corn require so much larger amounts of water it is evident if watersheds were entirely covered with them, that little water would be left for stream flow during the growing season.

In order to show the significance of these several demands for water during the growing season, the following discussion is submitted.

According to the United States census of 1880, Allegany county, which is, as indicated by the watershed map accompanying Mr. Bogart's report of 1890, about three-quarters in the Genesee river drainage area, has:

	Square miles.
A total area of	952.4
Total improved area	663.5
Tilled area	424.8
Permanent meadows	238.7
Total unimproved	288.9
Woodland and forest	253.5
Miscellaneous unimproved	35.4

The total area of the Genesee drainage area above the site of the proposed dam at Mount Morris is 1,060 square miles, and applying to this area the percentage of improved area, tilled area, permanent meadows, etc., as they existed in Allegany county in 1880, we have for the entire watershed:

	Square miles.
A total area, as already stated, of	1,060.0
Total improved area	738.4
Tilled area	472.8
Permanent meadows	265.6
Total unimproved	321.6
Woodland and forest	282.2
Miscellaneous unimproved	39.4

From what has preceded we derive the following estimate of average amount of water taken up by crops, etc., in the upper

Genesee river drainage area during the growing season from June to August, inclusive.

	Per cent of whole area.	Water required in the growing season. Inches on crop area.	Inches on whole area.
Tilled area	45	10.2	4.64
Permanent meadows	25	16.0	4.00
Woodland and forest	26	3.6	0.94
Miscellaneous unimproved	4	6.0	0.24
Totals	100	.....	9.82

Adding something to the total of 9.82 inches for the extra evaporation from crop areas, for decrease of timber area since 1880, and for other small losses, and we find that in the upper Genesee river drainage area growing crops will require probably about 11.0 inches of water over the whole area during the growing season from June to August, inclusive.

Referring to Table No. 7, we observe that the year has been divided into three periods, the first of which, from December to May inclusive, may be called the storage period. During these months vegetation is mostly inert and evaporation at a minimum, and as the result of continuous gagings on a number of streams, it is found that from 60 to 80 per cent of the rainfall runs off in the streams during this period. For the upper Genesee river it is considered that 70 per cent of the rainfall will run off in this period in the year of minimum precipitation.

With June begins the period of active growth, which continues through July and August. Examining Table No. 7, we learn that the rainfall during the growing season will not, generally speaking, be any in excess of the demands of water for vegetation. In the years of minimum rainfall it may even be one or two inches less. Comparative gagings have shown that in the years when rainfall is about equal to the demands of vegetation, etc., the run-off of streams is nearly constant at one and one-half inches on the whole drainage area. This amount is supplied almost entirely from the ground water stored during the previous storage period.

In years when the rainfall is less than these demands, the vegetation will draw to some extent on the ground water supply.

In the case of the upper Genesee the estimates of demands for plant growth clearly show an excess over the rainfall for average years. Hence it will not be safe to estimate the run-off at more than one inch over the whole area in the year of minimum flow.

September, October and November are the replenishing period. Nearly every demand for vegetation ceases by September first, while after October first the rapid fall in temperature tends to soon reduce evaporation to a very low quantity. The rainfall of this period goes largely to replenish the depleted ground water, and it is only after the ground water has reached its usual level that large run-offs occur; though on this point it may be remarked that the character of the rainfall, whether rapid or slow, will materially influence the quantity of the run-off in both the growing season and the replenishing period. Slow, drizzling rains in the growing season show absolutely no effect on the streams, while in the replenishing period, until after the ground water has reached its usual level, their effect is also very slight. A number of elegant illustrations of this general principle appear on the diagrams of flow of the Genesee river at Mt. Morris and Rochester, herewith submitted, and which are so clearly defined thereon as to render extended description unnecessary.

Applying the data derived from other gagings, and it is considered unsafe to estimate the run-off of this period for the minimum year at more than nine-tenths of an inch.

Referring to column 21 of Table No. 7, where the assumed safe amounts collectible in the year of minimum rainfall are carried out, and we find that the total amount collectible from the upper Genesee area, in the year of minimum rainfall, may be placed at 10.60 inches over the whole area.

Of all the rainfall records included in columns (1) to (18), inclusive, of Table No. 7, that at Rochester from 1829 to 1877, a period of about 45 years, is the longest. This record shows, in common with a number of the others, that the rainfall of the growing

and replenishing periods is a little over nine inches, and while the mean annual rainfall of the upper Genesee is about five inches greater than that of the lower, a study of the record shows that the excess is mostly in the storage period. At Rochester, where the minimum annual rainfall is as low as 20 inches, it is doubtful if anything like 10.60 inches could be collected in the minimum year. Probably 8.50 inches would be a safer estimate there. The long Rochester record, however, by reason of agreeing with the records of the upper valley, for the growing and replenishing periods, seems to enforce the reasonableness of the conservative estimate of run-off for these two periods which has been made.

It may be noted, however, that severe storms in the growing and replenishing periods may, at times, add something to the flow of a stream over and above that furnished by the ground water. In order to show the possible value of such accessions the greatest precipitation in twenty-four hours in each month, so far as available, has been included in Table No. 7. In the storage period, when the ground is frozen and covered with snow and ice, this part of the record is also of value as indicating the possible maximum flood of that period. This latter point will be touched upon further on. The effect of even severe storms in the growing period is comparatively slight, especially after the ground water has become depleted, as may be sufficiently illustrated on reference to the diagrams. But as soon as the ground water has reached its high level the effect of even slight storms is very marked.

Without going further into the detail, as a summation of the whole matter, it may be stated that the minimum flow of the upper Genesee may be taken at 10.60 inches for the whole year, with 8.70 inches in the storage period from December to May, inclusive, and the balance in the months from June to November, inclusive. For reasons in detail, other than those already given, reference is made to Mr. Vermule's discussion in the New Jersey geological reports, as already stated. As to the total amounts realized from the collection of different depths over the drain-

age area, reference may be made to the tabulated statement accompanying Mr. Kibbe's report of 1890.

If we consider the Genesee river as a whole we find that very different conditions govern. Between Rochester and Mount Morris and between Mount Morris and Dansville, in the broad valley of the Canesaraga creek there are extensive flats, amounting for the whole to perhaps 80 square miles. The drainage area at Rochester is 2,425 square miles as against 1,060 square miles at Mount Morris. The portion of the drainage area below Mount Morris also contains Honeoye, Canadice, Hemlock and Conesus lakes, which altogether provide a large volume of surface storage, while above Mount Morris there are few flats and only one small lake (Silver lake). The gagings made by Mr. Kibbie in 1890, as well as the gagings made since last August by the present writer, abundantly show that the upper section has a rapid run-off and is subject to sudden and excessive flood flows. These flood flows are received in the extensive flats below Mount Morris, where they are partially retained and gradually delivered to the extreme lower river. On inspecting the diagram, showing the relation of stream flow at Mount Morris and Rochester to rainfall and temperature, it is seen that the flood flows at Mount Morris are invariably as great as at Rochester, although the dry weather flow at Rochester is, proportionately to the drainage area, much greater than at Mount Morris. The flats then act to decrease the flood flow at Rochester and to increase the dry weather flow there. At Mount Morris we may expect flood flows of from 25,000 to 30,000 cubic feet per second nearly every year, while at Rochester 30,000 cubic feet per second is quite rare, even the great flood of 1865, probably did not materially exceed 35,000 cubic feet per second. About 30,000 cubic feet at Rochester gives a full river, and anything much beyond that figure will produce a disastrous flood. The flats then act to decrease in a very marked degree the violence of the spring freshet at Rochester. With the river in its natural state, and with the same character of drainage area throughout its whole course, that we find to exist above Mt. Morris, what is now the chief busi-

ness portion of the city of Rochester would certainly be entirely submerged nearly every year.

This immunity of the city of Rochester is, however, purchased at the expense of the 80 square miles of flats which annually act as an immense storage reservoir for the spring floods of the upper river.

From an economic point of view one marked effect of the annual inundation is to largely prevent the use of these flats for any agricultural purpose other than grazing. If they can be certainly relieved of the burden of that portion of the annual overflow which occurs in May, they will immediately become the most fertile agricultural lands in the State, and their value will be doubled. It is in line with the policy of all civilized governments to establish works for river conservancy wherever results are to be gained such as these, and the precedent of similar works by other governments is in view of the benefits to be derived by the commonwealth, in the way of increased valuation of property, the strongest possible argument that can be urged in favor of the Genesee river storage. The construction of the proposed dam to the height of 130 feet will so far control the flood flow of the stream as to almost entirely relieve the Genesee valley between Rochester, Mt. Morris and Dansville of the burden of that portion of the annual overflow which now tends to greatly decrease the value of these lands for any agricultural use other than grazing.

The question may be asked, whether the annual inundation is not really a benefit rather than an injury, by reason of carrying a large amount of valuable silt fertilizing material over the entire submerged area, as in the case of the river Nile and other irrigating streams. The answer is that, by reason of a heavy May rainfall, occuring at a time when the ground water is high and before vegetation has become active, there is likely to be an overflow just at the planting season, which effectually prevents the putting in of crops. Frequently, too, the May overflow extends over into the early days of June. On examining the diagram of river flow at Rochester, contributed to the survey by J. Y. McClintock, C. E., city surveyor of Rochester, it is seen that in 1893 the May overflow was at its extreme height on the

seventh, when the discharge at Rochester was at the rate of over 14,500 cubic feet per second; and on May twentieth the mean discharge at Rochester was 12,900 cubic feet per second. Flood discharges of these amounts are sufficient to render farming operations impossible on a considerable portion of the flat area. On June 2, 1889, the discharge at Rochester was at least 20,000 cubic feet per second, and from personal observations by the present writer on that day it is known that nearly the whole flat area of the valley was flooded. The answer to the hypothetical question as to the value of the annual overflow is, therefore, that in the case of the Genesee valley, the May overflow comes at such a time as to do only injury, without any opportunity to realize what would be, if the inundation came only in March or April, a great benefit.

The cash value then of so regulating the flow of the river as to do away with the May overflow, a result easily accomplished, can be estimated as an average of 80 square miles, at, say $40 per acre, or the increased valuation of the whole area would be about $2,050,000.

Moreover, the flats above Rochester are a further benefit to the lower river by reason of an immense storage of ground water therein, which, as the flood level subsides, gradually runs out with the result of greatly decreasing the period of extreme low water.

Again, in case of excessively heavy rains, in the middle of the summer, from the effect of which the river channel is temporarily, partially or wholly filled, such an amount of water is stored in these flats as to keep the river comparatively well up during the fall.

This actually happened in the season of 1893, when on August 29, there occurred a rainfall of nearly three inches over the whole drainage area in a period of about 12 hours, which produced a flood flow of 5,800 cubic feet per second at Mount Morris, and 4,800 cubic feet at Rochester; an amount of water sufficient to partly fill the channel between these two places, but without any overflow of the adjoining flats. Previous to this heavy rainfall, the mean flow at Rochester had been for a month about 460

cubic feet per second. At Mount Morris it had not averaged, for the same period, more than 125 cubic feet per second. The effect of this rain on the ground water of the flats, is strikingly shows by comparison of the two flows on the Mount Morris and Rochester diagram, where it will be seen that on September 3, the flow at Mount Morris, was again down to 200 cubic feet per second, and remained below that figure, except for slight rises due to rainfall on September 7 and 8, and September 15 and 18, until October 15, when the flow rose to a little over 2,000 cubic feet per second. At Rochester, on the other hand, the effect of the heavy rainfall of August 29, was to so far replenish the depleted ground water of the flats as, with the exception of a few days in the early part of October, when the flow dropped to about 800 cubic feet per second, to keep the flow up to about 1,000 cubic feet per second, for the balance of the year.

In order to further illustrate the great storage value of the flats, we may note that the drainage area at Rochester is 2.3 times that at Mount Morris; hence, for proportionate yields the flow at Rochester should be 2.3 times that at Mount Morris. From the diagrams we learn, however, that during August, 1893, at a time of extreme dry weather, the flow at Rochester was 3.7 times that at Mount Morris, and after the extreme storm of August 29, which replenished the ground water of the flats, the flow at Rochester, during the entire replenishing period (September, October and November), was more than five times that at Mount Morris.

A knowledge of this constant accession of large quantities of water from the flats leads to another conclusion of great practical importance, namely, that we may expect to realize, at Rochester, the full value of all the water added from the storage at Mount Morris; that is, an addition of, say, 700 cubic feet per second at Mount Morris, in time of low water, will be likely to increase the flow at Rochester 700 cubic feet per second more than it would have been without such addition.

In order to show more strikingly the value of the flats for such storage, we will now compute the amount stored and held back therein.

Referring to Rafter and Baker's Sewage Disposal in the United States, page 165, we find a tabulated statement of the per cent of empty space in a number of soils as follows:

	Per cent.
In Illinois prairie soil, the voids are	55.2
In East Winsdor, Connecticut, clay soil, the voids are...	48.3
In coarse river sand, the voids are from ..... 38.4 to	41.0
In subsoils, the voids are from ..... 34.6 to	42.6
In blowing sands, the voids are	44.7

From these figures we learn that an estimate of 33 per cent of void space in the soils of the flats would be very conservative. The mean low water surface of the river channel is mostly from 15 to 20 feet below the surface of the flats. We will also assume that the water runs out of the upper 5 or 6 feet quickly, but that it is retained and delivered slowly from the balance. We have then 33 per cent of say 12.0 feet or 4.0 feet in depth over 80 square miles as the probable available ground water storage of the flats. For 80 square miles this amounts to $(80 \times 640 \times 43{,}560 \times 4) = 8{,}921{,}088{,}000$ cubic feet. If there were any way to control this ground water storage of the flats, it would by itself furnish an outflow of 800 cubic feet per second for four months, or 130 days. The diagram herewith submitted, taken in conjunction with the preceding discussion, shows about what the effect of this ground water storage is in the average year. For instance, on referring to Mr. McClintock's diagram of flow at Rochester, it is learned that the effect of this storage in June and July of last year was very marked. From it we see that the rainfalls of these two months had almost no effect on the flow of the stream which must have been supported almost entirely from the ground water. From May 28 to July 24, the delivery of the stream was steadily downward.

During this period the rainfalls were all used up by the demands of growing vegetation, and the flow of the stream was that due to stored ground water only, except possibly a very slight effect from the rainfall on June 6. By July 24, what may be termed the high level rapid

run-off ground water of the flats was entirely exhausted, and from that time on the flow was merely due to the deeper seated ground water of the whole area, assisted, however, by the relatively more rapid delivery of the flats. It may be remarked that the surface storage of the lakes of the lower river system is usually about exhausted by July 24.

As to the propriety of including in this discussion the area of the Canesaraga flats it may be mentioned that high water is stated by the inhabitants to only occur there when the Genesee is full to overflowing and is therefore mostly the result of backwater from the Genesee. The drainage area of the Canesaraga creek is 259 square miles, and although the creek channel has for several miles only slight declivity, it probably has capacity enough to discharge the ordinary flood flows, provided the Genesee were kept within its banks.

It must not be overlooked, however, that in the absence of instrumental surveys anything said on this point is subject to modification.

As a final conclusion of this division of the discussion it may be said that the construction of a storage dam, 130 feet in height, in the manner and at the point proposed, would so far regulate the flood flow of the river as to prevent any disastrous overflow of either the flats or the exposed area in the city of Rochester.

With intelligent management the outflow would be so regulated as to run the river channel full, or nearly full (but without overflow of flats) up to about June 1. In this way danger of floods would be removed and the benefits of the ground water storage in the flats retained.

Again the storage dam would render possible the producing of an artificial overflow of the flats in April, which would give all the benefits of silt irrigation at a season of the year when there would be no injury to any one.

The storage dam would furnish means for controlling such an overflow, both as to depth of water over the flats and as to length of time it would be allowed to remain.

We will now discuss briefly the possible maximum flood flows of the river and the effect of the proposed storage reservoir in

mitigating their severity. The data for this portion of the discussion is scanty, that directly applying being confined to Mr. Kibbe's gaging of 1890, as given on a diagram accompanying his report of that year, and the gagings of last year made at Mt. Morris and at Rochester, the results of which are included on the two accompanying diagrams already discussed. As stated in the preliminary report, a determination of the maximum floods has a practical bearing on the length of spillway to be provided for the storage dam, and while we have only cursory knowledge of their magnitude, as derived from direct observation on the stream itself, we may learn something by considering the flows of other streams of similar topography and the same or nearly the same drainage area. The following are the only American streams of approximately the same drainage area of which fairly complete statements of extreme flood flows are at hand:

(1) The Passaic river, in New Jersey, with an area of watershed at Little Falls of 773 square miles. The highest recorded flood flow is that of September, 1882, at which time 19,105 cubic feet per second flowed. This would be at the rate of 24.7 cubic feet per second per square mile, which quantity applied to the upper Genesee drainage area of 1,060 square miles, gives 26,182 cubic feet per second. The Passaic, however, can not be taken as a safe guide in this particular, because it has, like the lower Genesee, a considerable area of flat and surface storage. As indicating the possibilities of extreme floods at Rochester, we find on applying a maximum flow of 24.7 cubic feet per second per square mile to the lower Genesee area of 2,425 square miles, a possible flood there of 59,898 cubic feet per second. Such a flood would sweep through a large area of the city, and, if long continued, would do damage amounting to hundreds of thousands of dollars. Inasmuch as the State has paid extensive damages at Rochester on account of the flood of 1865, such figures are suggestive as to the desirability of reasonable expenditure either to prevent a recurrence of the flood of 1865 or of the much larger one which a comparison of figures actually obtained from the Passaic shows to be among the possibilities.

(2) The Raritan river, in New Jersey, with a watershed above the point of gaging of 879 square miles. This stream has very

little storage anywhere on its watershed. The drainage area is mostly under high cultivation, only 13 per cent being in forest. From this fact its extreme flood flows are likely to be somewhat larger proportionately than those of the upper Genesee, with which in other particulars it is fairly comparable. There have been three great freshets on this stream during the present century, in 1810, 1865 and 1882. The following are some of the particulars of that of 1882:

On September 22, at 3 p. m., the flow was 7,000 cubic feet per second.

On September 23, at 3 p. m., the flow was 35,000 cubic feet per second.

On September 24, at 5 p. m., the flow was 52,000 cubic feet per second.

On September 25, at 7 a. m., the flow was 7,000 cubic feet per second.

Thus giving a total discharge in 64 hours of 6,489,000,000 cubic feet or 3.36 inches on the watershed.

The maximum on September 24 was 59.2 cubic feet per second per square mile, which figure, when applied to the upper Genesee, gives 62,752 cubic feet per second from the whole area.

(3) We come now to a case nearer our own, namely that of the great flood in the Chemung river at Elmira, in 1889, at which time, according to the thorough study of Mr. Collingwood, this stream gave a maximum flow from a drainage area of 2,055 square miles of 138,000 cubic feet per second, equivalent to 67.1 cubic feet per second per square mile, which quantity applied to the upper Genesee would give a possible maximum discharge of 71,126 cubic feet per second.

The Chemung drainage area joins the Genesee area on the east and an inspection of the State meteorological bulletin maps shows that the rainfall for the Chemung area must be about the same as for the upper Genesee. We may, therefore, consider the 1889 flood on that river a little in detail, the more especially since it occurred in the period of general heavy rainfall during which the dam at Johnstown broke, and at which time the Genesee

river at Rochester discharged 20,000 cubic feet per second as noted on a previous page of this report.

In the vicinity of Elmira, the rainfalls which produced this flood were as follows: At Wellsboro, 36 miles southwesterly, the total precipitation was 9.8 inches, of which 7.45 inches fell after 9 p. m., of May 31, and before 7 a. m., of June 1.

At South Canisteo, about 45 miles west from Elmira and only a few miles east of the dividing ridge between Chemung and Genesee drainage, the total rainfall was 6.25 inches, of which 4.6 inches fell in three hours. Two miles west of South Canisteo 6.0 inches fell in the same time.

At a number of other points in the vicinity, rainfalls of from 6.0 to 8.0 inches in not more than 10 to 15 hours were observed.

The Chemung river is formed by the junction, a short distance above Elmira of the Tioga, Canisteo and Cohocton rivers.

The Tioga is a rapid stream heading near Blossburg in Tioga county, Pennsylvania, and descends to its junction with the other streams, at rates varying from 80 feet per mile at its head to 11 feet per mile at its mouth.

The Canisteo, of which the drainage area adjoins the Genesee on the west, has slopes in its lower reaches of about 5.5 feet per mile. The Cohocton is also a stream of comparatively low slope, while the Chemung itself, below the point of junction of the several streams, has, for some distance, a slope of 5.9 feet per mile, although through the city of Elmira, the slope is only 3.5 per mile.

We have then the Tioga, a rapid stream of 750 square miles drainage area; the Canisteo much less rapid, with a drainage area of 780 square miles; the Cohocton, also, only moderately rapid, with an area of 425 square miles, and the Chemung directly above Elmira, draining about 100 square miles, making a total of 2,055 square miles, as already noted.

The net results of a rainfall at the end of May, when the ground water was high, of from 6.0 to 10.0 inches over an entire area, varying in composition in the manner indicated, was a flood of such unprecedented magnitude that its like may probably not occur again in a century, and the practical question is whether the spillway of the Genesee storage dam should be designed with

reference to the possible extreme. Taking into account the character of the rainfall — that is, its distribution and the tendency to heavy falls in 24 hours, the character of the drainage area, the data derived from studying the flood flows of other streams of which the drainage areas are known, and some of which have been cited in the foregoing, and the fact that the present dam at Mount Morris, with a spillway 337 feet in length and 10 feet from crest of spillway to top of parapet wall, has never yet, in 50 years service, been taxed beyond its capacity to discharge, it has seemed to the present writer that the length of spillway of 400 feet, tentatively assumed for the purpose of the preliminary report, could be fairly settled upon as sufficient for any probable demands. In arriving at this conclusion it has been further assumed that, inasmuch as the extreme flood flows are of short duration, a maximum depth on spillway of 10 feet could be allowed. This gives for 400 feet length a discharge of about 41,000 cubic feet per second, equivalent to 38.8 cubic feet per second per square mile.

It may be further stated that before arriving at the foregoing conclusion, so far as the writer knows, all the American literature of stream flow in relation to rainfall has been examined. Its voluminousness, however, prevents more specific reference in this place.

As a final question we will consider in detail the effect of the storage provided by a dam 130 feet in height in preventing disastrous overflows in the broad valley below.

In the first place the storage provided by such a dam will be, as shown by Mr. Bailey's tabulation, accompanying his report of 1890, from 7,040,000,000 to 7,670,000,000 cubic feet, the exact quantity depending upon whether Site No. 1 or No. 2 is finally selected; 7,500,000,000 cubic feet would amount to nearly 3 inches over the whole drainage area.

On examining the records of greatest rainfalls in 24 hours, in columns (2), (6), (7), (8), (9), (10), (11), (14), (15) and (16) in Table No. 7, we find that we are likely to have two inches of precipitation in 24 hours during the months of December, January, February and March, at which season we may further expect to have

at times the ground frozen, and at the same time a considerable body of snow, which may likewise be melted by a heavy rainfall. We may assume the contribution from such source sufficient to make a run-off of 3 inches in 24 hours, or enough to completely fill the reservoir created by a dam 130 feet in height. If such a run-off occupied three days time it would be at the mean rate of about 29,000 cubic feet per second.

The management of the reservoir would ordinarily be such as to leave it empty at the end of January. If then the reservoir were to be completely filled during February or March by such an extreme inflow as we have just discussed, the natural course would be to gradually draw it well down through the sluices, leaving ample space for storage of the heavy flow of May. Working on these lines probably the lower Genesee would not go out of its natural channel more than once or twice in a century.

The most unfavorable case that can be assumed is that of the occurrence of an extreme run-off when the reservoir is full to the flow line. Even under such circumstances the reservoir will still act as a great mitigator of an extreme flood flow, as may be seen by inspecting Table No. 8, which has been prepared specially to illustrate the point in question. The following discussion will indicate the principle embodied in this table.

As already stated, the efficiency of the proposed storage reservoir as a flood moderator will depend upon the storage capacity in relation to the quantity of water flowing in from the drainage area. This capacity includes all storage space, whether above or below the crest of the overflow weir, which may be available at any time of heavy storm. Water is stored in the space above the crest only temporarily, but this space may still play an important part in reducing the maximum discharge below the reservoir, by extending the time within which the total surplus has to be passed down.

Inasmuch as extreme flood flows are of short duration, we may neglect the effect of evaporation, absorption and leakage, whence it becomes evident that the discharge by the overflow weir or sluices will be equal to the quantity received, less the quantity retained, whether temporarily or otherwise.

TABLE No 8.—EFFECT OF FULL RESERVOIR IN MITIGATING EXTREME FLOODS.

h. above crest in feet.	Total depth in reservoir in feet.	Storage =C. in cu. ft.	Intermediate values of C.	Q at height h. cu.ft. per sec.	Qp= means values of Q cu.ft. per sec.	Time t. in seconds.	Total time in seconds.	Outflow in each period in cu. ft.	Total outflow in cu. ft.	Total inflow in cu. ft.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
0	130.0	0	.....	0	.....	0	0	0	0	
			51,221,000		230					
0.5	130.5	51,221,000	.....	459	.....	1730	1720	395.700	395,700	
			51,297,000		899					
1.0	131.0	102,518,000	.....	1340	.....	1763	3483	1,583,800	1,979,500	
			51,374,000		1864					
1.5	131.5	153,892,000	.....	2388	.....	1826	5309	3.408,500	5,388,000	
			51,449,000		3082					
2.0	132.0	205,341,000	.....	3677	.....	1908	7217	5,784,400	11,167,000	
			51,526,000		4406					
2.5	132.5	256,767,000	.....	5139	.....	2018	9230	8,874,900	20,042,300	
			51,602,000		5947					
3.0	133.0	308,369,000	.....	6755	.....	2145	11375	12,757,700	32,800,000	
			51,678,000		7634					
3.5	133.5	360,047,000	.....	8512	.....	2311	13686	17,640,000	50,440,000	
			51,754,000		9456					
4.0	134.0	411,801,000	.....	10400	.....	2519	16205	23,822,000	74,262,000	
			51,830,000		11405					
4.5	134.5	463,631,000	.....	12410	.....	2787	18992	31,790,000	106,052,000	
			51,907,000		13470					
5.0	135.0	515,538,000	.....	14530	.....	3140	22132	42,298,000	148,350,000	
			51,983,000		15655					
5.5	135.5	567,521,000	.....	16780	.....	3624	25756	56,730,000	205,080,000	
			52,059,000		17943					
6.0	136.0	619,580,000	.....	19106	.....	4318	30074	77,478,000	282,553,000	
			52,135,000		20335					
6.5	136.5	671,715,000	.....	21569	.....	5396	35470	109,035,000	392,295,000	
			52,212,000		22823					
7.0	137.0	723,927,000	.....	24076	.....	7275	42745	166,035,000	558,330,000	
			52,287,000		25389					
7.5	137.5	776,214,000	.....	26702	.....	11340	54085	287,902,000	846,332,000	
			52,364,000		28059					
8.0	138.0	828,578,000	.....	29416	.....	26980	81068	756,972,000	1,603,304,000	2,431,782,000
			52,439,000		30816					
8.5	138.5	881,017,000	.....	32216	.....	.....	.....	.....	.....	
			52,516,000		33658					
9.0	139.0	933,533,000	.....	35100	.....	.....	.....	.....	.....	
			52,592,000		36583					
9.5	139.5	986,125,000	.....	38065	.....	.....	.....	.....	.....	
			52,668,000		39588					
10.0	140.0	1,038,793,000	.....	41110	.....	.....	.....	.....	.....	

We will assume that the water stands at the level of the crest at the instant when the inflow becomes equal to 30,000 cubic feet per second, and that the inflow remains constant at that figure for 24 hours, after which it gradually decreases. We desire to determine the length of time which will elapse before the outflow reaches 30,000 cubic feet per second, and the approximate time it will remain at about that figure. With the following notation:

h = any given height above the crest in linear feet and h_1, h_2, h_3, etc., successive equal heights.

c = storage capacity corresponding to h, and c_1, c_2, c_3, etc., successive capacities.

q = discharging capacity of the overflow weir in cubic feet per second, as determined by the formula $Q = 3.3 \sqrt{h^3} \times l$ for the given values of h, h_1, h_2, h_3, etc.

Q_p = the mean discharge in cubic feet per second for any given period, as for instance, $Q_p = \frac{Q_{h_2} + Q_{h_3}}{2}$ and $Q_{p_1} = \frac{Q_{h_2} + Q_{h_3}}{2}$, etc.

S = inflow from drainage area, taken in the present case at 30,000 cubic feet per second; and

t = the time in seconds in which the water will raise to any given value of h above crest.

Whence we have the formula,

$$t = \frac{c}{S - Q_p}$$

by which Table No. 8 has been computed.

On referring to Table No. 8 we learn:

(1) That, with water surface in reservoir at level of crest of overflow weir and a constant inflow of 30,000 cubic feet per second, it will be about 6.5 hours before the outflow will reach 15,000 cubic feet per second.

(2) That under the same conditions it will be about 24 hours before the outflow will reach approximately 30,000 cubic feet per second.

(3) Inasmuch as the original assumption was that the inflow should only be at the rate of 30,000 cubic feet per second for 24 hours and then gradually decrease, we may therefore say that the flow at rate of about 30,000 cubic feet per second would only be for say two or three hours, instead of at least 24, as it would have been without the assistance of the surface storage of the reservoir.

(4) The total inflow in 22.5 hours would be 2,431,782,000 cubic feet, of which 34 per cent of the whole would be stored during that time temporarily in the reservoir.

Other deductions can be made, but the foregoing are enough to show the great practical value of such a reservoir as a moderator of floods even when entirely filled at the beginning of the maximum flow.

In the same way if we assume the reservoir full and an inflow at the rate of 40,000 cubic feet per second, we learn on making the numerical computation that about 19 hours would elapse before the outflow would reach approximately that amount, in which time a depth of ten feet would be reached on the crest. The total inflow in 19 hours would be 2,904,735,000 cubic feet, of which 1,865,942,000 cubic feet would flow out and 1,038,793,000 cubic feet, or nearly 36 per cent of the whole, would be stored temporarily on the surface of the reservoir.

On referring to page 96, the statement will be found that flood discharges at Rochester, of from about 13,000 to 14,500 cubic feet per second, and from these quantities up to 20,000 cubic feet per second, represent more or less flooding of the flats above Rochester. This statement must not be taken to mean that, when such flows take place at Rochester, the inflow to the flats is anything like as small as 15,000 to 20,000 cubic feet per second. Table No. 8, may serve to enforce the statement that the inflow to the flats from the whole tributary area may be, at times, as large as 100,000 cubic feet per second, and there still be no serious flood at Rochester.

Making a rapid resume of the whole matter, and we may say that the benefits to be derived from the construction of the proposed Genesee river storage, are:

(1) The furnishing of an adequate supply of water to the eastern section of the western division of the Erie canal, under any and all circumstances.

(2) A great increase in the permanent water power of the Genesee river at Mount Morris, Rochester and intermediate points.

(3) The protection of the Genesee flats and the city of Rochester from destructive floods.

If we compare the estimated cost of the proposed storage with the cost of other large storage projects, we find that the Genesee storage will compare very favorably with other large projects of this character thus far carried out, as, for instance:

(1) The Sweetwater masonry dam in California, where 784,080,000 cubic feet are stored, cost $938.70 per million cubic feet stored.

(2) The Hemmet Valley masonry dam in California, where 6,111,280,000 cubic feet are stored, cost $229.10 per million cubic feet stored.

(3) The Periar masonry dam in India, where 6,969,600,000 cubic feet are stored, cost $106.75 per million cubic feet stored.

(4) The Betwa masonry dam in India where 1,603,008,000 cubic feet are stored, cost $204.25 per million cubic feet stored.

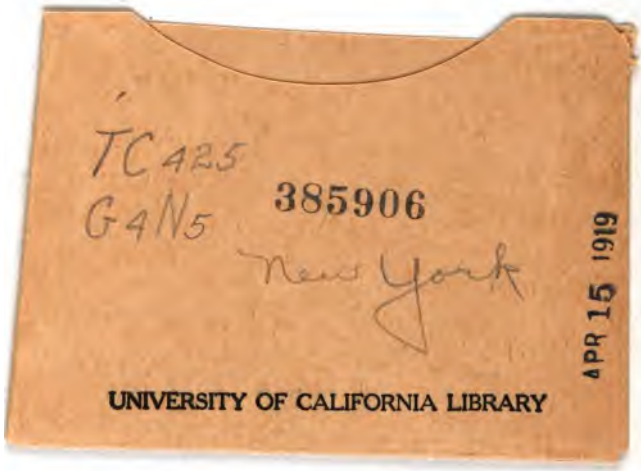
Taking the total cost of the Genesee river storage dam, 130 feet in height, at $2,400,000, and the storage at 7,700,000,000 cubic feet, and we derive a cost per million cubic feet stored of $311.69.

On taking the total cost at $2,600,000, and the storage at 7,100,000,000 cubic feet, and we have a cost per million cubic feet stored of $366.20.

Respectfully submitted.

GEORGE W. RAFTER,
Engineer in Charge.

YC 69051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